STRESS-STRAIN-TIME BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED MARINE CLAY UNDER VARIOUS STRESS PATHS

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By

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to the

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TO MY MOTHER

CERTIFICATE

This is to certify that the thesis entitled "Stress-Strain-Time Behaviour of Anisotropically Consolidated Marine Clay Under Various Stress Paths", by Mr. S.K. Mathur, for the award of the Degree of Doctor of Philosophy, of the Indian Institute of Technology, Kanpur is a record of bonafide research work carried out by him under my supervision and guidance. The results embodied in this thesis have not been submitted to any other University or Institute for the award of any degree or diploma.

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NOTATION

A	-	stress path and parameter of Berkeley model;
A_1, A_2, A_3	-	constants;
a_2	-	constant;
B	-	stress path;
B'	-	constant;
C	-	stress path;
C'	-	constant;
C_α	-	coeff. of secondary compression;
$C_{\alpha\varepsilon}$	-	slope of linear portion of percent shear strain vs. log time plot;
$C_{\alpha\varepsilon_1}$	-	slope of linear portion of percent axial strain vs. log time plot;
$C_{\alpha\varepsilon_1}$ (A)	-	$C_{\alpha\varepsilon_1}$ for stress path A;
$C_{\alpha\varepsilon_1}$ (C)	-	$C_{\alpha\varepsilon_1}$ for stress path C;
$C_{\alpha v}$	-	slope of linear portion of percent volumetric strain vs. log time plot;
D	-	stress path and $(\sigma_1 - \sigma_3)$ as used in Berkeley model;
D'	-	constant;
D_{max}	-	$(\sigma_1 - \sigma_3)$ at failure (Berkeley model);
$\bar{D}$	-	D/D_{max} (Berkeley model)
E	-	stress path;
E_1	-	stress path;
E'	-	Effective Young's modulus;

E'_{iA}	-	Effective ~~initial~~ tangent Young's modulus (stress path A);
E'_{iC}	-	Effective initial tangent Young's modulus (stress path C);
e	-	void ratio;
F	-	stress path;
F_1	-	stress path;
G	-	stress path;
G_1	-	stress path;
G'	-	Effective shear modulus;
H	-	depth of clay layer;
K'	-	Effective bulk modulus;
K_o	-	coefficient of earth pressure at rest;
L	-	length of sample;
LL	-	liquid limit;
M	-	critical state frictional constant;
m	-	creep potential (Berkeley model);
NCC	-	normally consolidated clay;
OCC	-	overconsolidated clay;
OCR	-	overconsolidation ratio;
P	-	mean pressure;
P'	-	effective mean pressure ($\frac{\sigma'_1 + 2\sigma'_3}{3}$);
P_c	-	critical pressure;
P_e	-	Hvorslev effective pressure;
P_o	-	initial pressure;

P'_o	-	initial effective pressure;
PI	-	plasticity index;
PL	-	plastic limit;
q	-	deviator stress $(\sigma_1 - \sigma_3)$;
q_f	-	deviator stress at failure;
t	-	time;
t_1	-	time (Berkeley model);
V	-	volume of the sample;
V_o	-	initial volume of the sample;
$\dot{v}$	-	change in volumetric strain;
W	-	dissipated power;
w	-	water content;
α_1, α_3	-	constants;
α_1', α_3'	-	constants;
$\bar{\alpha}$	-	parameter of Berkeley model;
ε	-	shear strain $[\frac{2}{3}(\varepsilon_1 - \varepsilon_3)]$;
ε_1	-	axial strain;
ε_3	-	lateral strain;
ε_o	-	initial value of ε_o;
$\dot{\varepsilon}$	-	change in shear strain;
$(\varepsilon^P)_{v^P}$	-	plastic component of shear strain (corresponds to an undrained test);
η	-	stress ratio (q/P');

η_o	-	initial value of η ;
η_{k_o}	-	η corresponding to k_o ;
θ	-	angle defining probes in $\sigma_1' - \sigma_3'$ space;
K	-	slope of swelling curve in e-ln P plot;
λ	-	slope of virgin consolidation curve in e-ln P plot;
ν'	-	effective Poisson's ratio;
ν_A', ν_C'	-	effective Poisson's ratio for A and C stress paths respectively;
$\rho_{cs'}$	-	secondary creep settlement rate;
$\sigma_1, \sigma_2, \sigma_3$	-	major, intermediate and minor principal stresses;
$\sigma_1', \sigma_2', \sigma_3'$	-	effective major, intermediate and minor principal stresses;
τ	-	shear stress;
ϕ'	-	effective angle of internal friction
$d\varepsilon_1$, $d\varepsilon_2$, $d\varepsilon_3$	-	increment in major, intermediate and minor principal strains;
$d\eta$	-	increment in η ;
dP,dq	-	increments in P and q;
dV	-	increment in volumetric strain;
dV^e, dV^P	-	increments in elastic and plastic components of volumetric strains;
$d\omega$	-	increment in ω ;
ΔL	-	increment in length;

ΔP	-	increment in pressure;
ΔV, δV	-	increments in v;
$\Delta\varepsilon$, $\delta\varepsilon$	-	increments in ε ;
$\delta\varepsilon^{P}$	-	plastic shear strain increment;
$\Delta\varepsilon_1$, $\Delta\varepsilon_2$, $\Delta\varepsilon_3$	-	increments in the three principal strains &
$\Delta\sigma_1'$, $\Delta\sigma_2'$, $\Delta\sigma_3'$	-	increments in the three effective principal stresses.

SYNOPSIS

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STRESS-STRAIN-TIME BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED MARINE CLAY UNDER VARIOUS STRESS PATHS

In the study of the stress-strain-time behaviour of the clay, resedimented samples have been anisotropically consolidated along two stress ratios and then sheared along various stress paths in the triaxial, covering almost the entire stress space. Stress-strain-volume change curves for different stress paths, both for normally consolidated and lightly overconsolidated samples have been presented. Anisotropic consolidation tests show dependence of λ on η. Unloading behaviour along various stress paths has been presented. Elastic moduli for various stress paths have been worked out using theory of elasticity and the limitations of this theory resulting in an anamolous behaviour for a few stress paths is pointed out. It is shown that the effect of anisotropic consolidation and light overconsolidation on modulus is marked. Tests along various stress paths on lightly overconsolidated samples have shown that the insitu behaviour of soft marine clays may, atleast qualitatively be predicted from such tests.

Drained triaxial creep behaviour has been studied along various stress paths and the results have been presented in the form of percent strain vs. log time relationships for normally consolidated and lightly overconsolidated samples. It has been shown that the curves, after a few hundred minutes of primary consolidation, resolve into linear strain-log time relationships. It is pointed out that the axial and shear logarithmic creep rates very much depend on the stress path and the stress ratio. Volumetric logarithmic creep rates are independent of stress ratio, whereas stress path seems to slightly affect volumetric creep rates as well. It is shown that the logarithmic creep rates for lightly overconsolidated samples are far too low as compared to the corresponding rates for the normally consolidated samples. The effect of aging on the logarithmic creep rates is marked.

The prediction of the stress-strain-time behaviour as experimentally observed in this investigation, has been attempted by means of a semi-empirical model. It is proposed that the test results of a wide variety of stress paths, may be predicted from the results of two basic tests: (i) consolidation (q-constant) test and (ii) pure shear (P-constant) test. The parameters for this model have been evaluated from the results of B, E, E_1, and B-unloading stress path tests. The stress level and the nonlinearity has been taken into account in the evaluation of the parameters. The predictions from

the proposed model have been compared with the other available models. The proposed model shows very good agreement with the experimental results for loading stress paths. It has also been suggested that the model can give reasonable predictions of k_o-value, the behaviour during undrained shear and the k_o-unloading.

The prediction of the drained creep rates by the proposed model compares very well for the two loading stress paths. The proposed semi-empirical approach is shown to be highly versatile. It has been suggested that the proposed model can be readily used to predict the settlements and the creep rates of footings on soft clays on the lines of Lambe's stress path method.

Berkeley rate process model parameters have been evaluated from the drained creep tests for various stress paths and it is suggested that this model, which was so far tried only for undrained creep, may be used to predict the steady state drained creep rates, provided an average 'm' value and '$\bar{\alpha}$' and 'A' as functions of stress path and stress history are used in the three-parameter equation.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Soil engineering requires the prediction of stresses and displacements induced in the ground due to any activity in the soil mass; it may be either the construction of a structure over it, or an excavation being planned for some purpose. The importance of the stress-strain behaviour of soil has been mainly realized after Roscoe's (1970) Rankine lecture and since then more & more effort is being made to obtain a constitutive relation, which can predict the soil behaviour in general. Before any such relation could be developed, a quantitative understanding of the mechanical behaviour of soils is very necessary.

1.2 STRESS-STRAIN-TIME BEHAVIOUR

It will be highly desirable to study the stress-strain-time behaviour insitu, however, it requires very sophisticated instruments to be developed. Although, quite a few instruments are being used for various tests, these are still in a developing stage. Because of these limitations, laboratory studies have to be carried out on the undisturbed samples. As the undisturbed samples are generally non-uniform, any check on the predictive power of a model, would be

masked due to this factor. It is, therefore, very desirable to study the basic behaviour on uniform laboratory samples prepared by resedimentation. Again, most of the studies have been made on isotropically consolidated samples. However, to get a realistic assessment of the field behaviour, tests which duplicate the insitu condition should be conducted. As there have been only a few investigations pertaining to relevant drained stress-strain-time behaviour, this important aspect has been considered in detail in this investigation.

1.3 PREDICTION

The models available for predicting the behaviour, are based on theory of elasticity, theory of plasticity, theory of visco-elasticity etc. The predictions from these models have usually been checked from tests conducted on isotropically consolidated samples, probably because a comprehensive investigation was not available on k_o-consolidated samples. However, a few recent studies on samples having k_o-consolidation history have been made and they indicate that the available models probably cannot predict this behaviour. Once a model is checked against its predictions on uniform lab. samples, it can then be thought of being applied, with caution, to very complex field conditions. The present study, has been conducted keeping this view point in mind.

1.4 PRESENTATION

The relevant literature associated with the mechanical behaviour mainly under triaxial test conditions and the various available models have been discussed in Chapter 2.

Chapter 3 gives the details of the experimental setup, soil type and test conditions along with the brief procedure of conducting the tests.

In Chapter 4, Stress-strain behaviour of anisotropically consolidated samples under various stress paths has been presented. Tests on normally consolidated and lightly overconsolidated samples are compared. Anisotropic consolidation results are given. Use of theory of elasticity in evaluating the elastic parameters with the help of the relevant tests is critically discussed.

Chapter 5 deals with the drained creep behaviour along various stress paths. The influence of stress path & stress ratio on the drained creep rates are discussed both for normally and lightly overconsolidated samples.

In Chapter 6, a model is proposed to predict the observed stress-strain-time behaviour of clays. Evaluation of parameters for this model is discussed. Predictions from the model are compared with those from other available models. The versatility

of the model is illustrated by predicting k_o, state boundary surface etc.

Chapter 7 gives the summary of the investigations and the conclusions on the different aspects of the study. Suggestions for further research are mentioned.

The list of references then follow Chapter 7.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The study of the stress-strain-time behaviour of soils was almost ignored so far, as practically all the research was aimed at obtaining their strength characteristics. It was Roscoe (1970),who in his Rankine lecture, has emphasized : "we should stop concentrating attention only on the shear strength of soils and think in terms of their stress-strain behaviour, especially at stress-levels corresponding to working loads which will probably be less than half the values required to produce failure". Attempts to separate a real soil response artificially into either an over-simplified deformation situation without regard to localized yielding which occurs, or into a failure problem which disregards the deformation in the soil, lead to many difficulties in the analysis, understanding and interpretation of the behaviour (Scott & Ko, 1969).

To obtain a unique constitutive relation, which can be applied to all types of soils and under various conditions, would be almost an impossible task. This is because of the extremely variable nature of soil deposits found in nature, which have been influenced by the vagaries of nature for millions of years. Insitu testing is the prime necessity (Lambe, 1973) of the day to evaluate the true field response. However, laboratory studies on the

undisturbed samples have been found to predict the insitu behaviour reasonably well (Bjerrum, 1973). The laboratory studies on resedimented samples have their own merit in a fundamental study, where the predictions of different models have to be checked on uniform samples, before its use to predict the field behaviour can be assessed. Their have been three stages of development :

(i) Engineering or macroscopic research,

(ii) Mathematical research in the field of continuum mechanics using the theories of elasticity, plasticity, visco-elasticity etc. and

(iii) Physico-chemical and microscopic research.

Physico-chemical and microscopic research has been the subject of study by many researchers (e.g., Lambe, 1951, 1953; Roscoe, 1967; Morgenstern and Tschalenko, 1967) and will not be discussed here. The first two approaches will be covered in short.

2.2 ENGINEERING OR MACROSCOPIC BEHAVIOUR

In the early years of development Hvorslev's (1937,1960) contribution along with the work of Bjerrum (1954) and Gibson (1953) on the fundamental shear strength parameters marked the beginning of the subsequent studies on the strength characteristics of the soils. The prediction of the failure strength and failure void ratio from a knowledge of the initial void ratio and the stresses was attempted by Rendulic (1936,1937). His findings were substantiated and elaborated by Henkel (1956, 1959, 1960). Recently, Varadarajan (1973) has given the concept of critical stress path and critical

OCR in re-establishing and rationalising the useful concepts laid down by Rendulic and Henkel.

Studies on the stress-strain behaviour of soils gained momentum with the advances in numerical techniques. A detailed investigation on the undrained stress-strain behaviour was given by Ladd (1964). Since then, there have been numerous investigations on the study of the undrained stress strain behaviour with special emphasis on the undrained modulus. See Ladd (1964, 1965), Janbu (1963), Crawford (1959), Casagrande and Wilson (1951), Richardson and Whiteman (1963), Ward, Samuels and Butler (1959), Cooling and Skempton (1942), Bjerrum (1964), Varadarajan (1973).

Compared to the study of undrained modulus, drained modulus has not been investigated to that extent. Effect of stress path on the drained modulus was emphasized by Lambe (1964, 1967). Yudhbir and Varadarajan (1975) have studied the dependence of drained modulus on stress path (four stress paths have been investigated) and confining pressure. That drained modulus is very much a function of stress path and confining pressure has been clearly brought out. They have solved a retaining wall problem, where the effect of using relevant stress path dependent moduli for each soil element on the deformations, has been clearly shown. A comprehensive review of the factors affecting undrained and drained moduli is given by Yudhbir et. al. (1975).

Although, Yudhbir and Varadarajan (1975) and Yudhbir et. al. (1975) have convincingly shown the dependence of stress path on modulus, the drained tests described by them are on isotropically consolidated samples. It is now well known that the modulus values obtained from tests on isotropically consolidated samples are far too low as compared to the actual insitu values. Thus, for a realistic assessment of the insitu stress-strain behaviour, the samples should first be subjected to the k_o-consolidation history before shearing along a relevant stress path. This has been emphasized by Berre and Bjerrum (1973), Vaid and Campanella (1974), Simons and Som (1969), Simons (1972), Bjerrum (1973) and Yudhbir et. al. (1975).

2.2.1 Field Behaviour

The field behaviour of soft marine clays is altogether different from what is obtained from the usual laboratory tests. The normally consolidated marine clays exhibit light overconsolidation insitu due to various factors such as delayed consolidation etc. (Bjerrum, 1967). It has been shown by Bjerrum (1973) that the insitu conditions can be restored if the undisturbed samples are reconsolidated to the actual stresses existing in the field before shearing.

Recently, Parry and Nadarajah (1974) have shown that atleast qualitatively, the insitu behaviour of soft clays can be simulated in the laboratory by conducting tests on resedimented lightly overconsolidated samples (OCR from 1 to 2.5). Undrained

tests conducted from various OCR values showed a distinctly different behaviour as compared to the corresponding behaviour observed for normally consolidated samples. This approach is very useful in simulating the insitu behaviour of soft clays in the laboratory.

From these studies as discussed, it is seen that barring the work of Yudhbir and Varadarajan (1975) on isotropically consolidated samples and that of Simons and Som (1969) and Vaid and Campanella (1974) on k_o-consolidated samples, hardly any data is available in the literature, where the variation of drained modulus is shown with stress path on k_o-consolidated samples. Also practically no data is available on the important aspect of the simulation of the drained field behaviour of soft marine clays in the laboratory. Hence drained triaxial tests along various stress paths are required for k_o-consolidated samples (both normally and lightly overconsolidated) to obtain the variation of modulus with stress path.

2.3 STUDIES OF DEFORMATION USING THEORY OF ELASTICITY AND PLASTICITY etc.

Soils have been studied as a discrete system consisting of an assemblage of particles by Rowe (1962), Horne (1965) and Barden and Khayatt (1966). However, it has been more common to consider soils as a continuum and to use the methods and results of continuum mechanics for studying the mechanical properties of soils.

2.3.1 Elasticity Methods

The simplest approach is to use the theory of linear elasticity. There is complete uncoupling of the hydrostatic and deviatoric components i.e., the volumetric strains are related to hydrostatic (or octahedral normal) stress by the bulk modulus and the deviatoric (or octahedral shear) strain is related to deviatoric (or octahedral shear) stress by the shear modulus. Because of linearity, superposition is applicable. No irrecoverable (plastic) deformation is described by this theory. A complete description of the mechanical properties by this theory requires only two parameters.

On the other hand, anisotropic theory of elasticity requires five constants from relevant tests. Use of theory of elasticity in various soil mechanics problems has been very well documented by Poulos and Davis (1974). Gibson (1974) in his Rankine lecture has shown the extent of modification of the predictions by elastic theory, if heterogeneity and anisotropy are taken into account in its formulation. Application of theory of anisotropic elasticity to over-consolidated clays has been indicated by Simons and Som (1969), Henkel (1972) and Wroth (1972).

In order to include both displacement and failure in analyses, Duncan and Chang (1970) developed simple, non-linear stress-strain relations based on the generalized Hooke's law and the Mohr-Coulomb failure criterion. The stress-strain curves were described

by hyperbolic relations following the developments of Kondner (1963). These relations were used for incremental analysis of embankments, excavations, and locks (Chang and Duncan, 1970; Clough and Duncan, 1971; Kulhawy, Duncan and Seed, 1969 and Kulhawy and Duncan, 1970). Yudhbir and Varadarajan (1975) have used stress path dependent moduli for an earth pressure problem.

Yudhbir et. al. (1975) have worked out the elastic parameters for four different stress paths using theory of elasticity and have shown that anomalous behaviour is obtained for stress path 'B' (p-constant test) and those in its neighbourhood. They suggest an alternative to circumvent this problem. However, these tests were conducted on isotropically consolidated samples, which obviously will not represent the insitu field behaviour.

Recently Domaschuk and Valliappan (1975) have analysed a settlement problem by using G and K parameters in the finite element programme, as obtained from p-constant and isotropic consolidation tests respectively. They have taken into account the effect of pressure and non-linearity of the stress-strain curve and their settlements show good agreement with the measured values. However, the author feels that this agreement may be fortuitous as first of all P-constant and consolidation tests from k_o- line should have been conducted (modulus values are erroneous for tests on isotropic samples , as discussed in section 2.2) and secondly, using parameters

G and K, as such, would result in decoupling of the strains, which is contrary to the observed soil behaviour.

It has been observed experimentally that (i) many soils are inelastically deformed almost immediately upon application of stress, (ii) there is coupling between volume changes and changes in shear stress, (iii) dense soils expand in volume during pure shear and (iv) many soils, drained or undrained, experience a decrease in strength with further straining after a peak strength has been reached. For these and many other reasons (like stress path dependent behaviour etc), elasticity theory or extensions thereof cannot be expected to account for the soil behaviour properly. Prévost and Höeg (1975) suggest that based on analytical studies with more rigorous and realistic stress-strain formulations, it should be possible to determine for what types of problems and soils, simple elasticity theory may give adequate predictions (also see Duncan and Chang, 1972 and Scott and Ko, 1969). It is desirable that elastic parameters be evaluated for other stress paths (typical footing stress paths etc.) from tests on anisotropically consolidated samples.

2.3.2 Plasticity Methods

There are three basic requirements for a plastic stress-strain theory : (i) there must exist a yield surface such that if the soil is subjected to changes in stress represented by points inside the surface, the soil will deform elastically, whereas if

the changes in stress cross the yield surface, it will yield plastically. The yield surface expands as the soil is loaded to successively higher stress levels, and at failure the yield surface coincides with the failure surface. (ii) a flow rule is required to relate the relative magnitudes of the strain increments to the stresses and (iii) a work-hardening law is needed from which the magnitudes of the plastic strain increments caused by a given stress increment can be determined. A brief review is given of the various models based on this approach.

2.3.2.1 Cam Clay Model

Investigations into the deformational behaviour of cohesive soils date back to Rendulic (1936, 1937) who established that for triaxial conditions (σ_1', $\sigma_2' = \sigma_3'$), the state of a normally consolidated element at all times during the loading process formed a unique surface in (σ_1', σ_3', ω) space. Almost concurrently, Hvorslev (1937) concluded that the peak shear stress at failure of a soil is a function of the effective normal stress, and of the voids ratio, in the plane of failure at the moment of failure, and this function is independent of the stress history of the sample. If Hvorslev's equation is plotted three dimensionally, it will define a unique surface in (σ', e, τ) space, which is called Hvorslev surface (Roscoe et. al., 1958).

Roscoe et. al.,(1958) combined the approaches of Rendulic and Hvorslev and could succeed in presenting a unified picture by plotting triaxial test results in three dimensional (P', q, ω) space, where

the hydrostatic stress component P' was defined as $(\frac{\sigma_1' + 2\sigma_3'}{3})$ and the deviatoric stress component q as $(\sigma_1' - \sigma_3')$. This work of the Cambridge group was subsequently refined and developed and is described by Roscoe, Schofield and Thurairajah (1963), Roscoe and Poorooshasb (1963), Calladine (1963), Roscoe and Schofield (1963), Poorooshasb and Roscoe (1963), Roscoe and Thurairajah (1964), and Schofield and Wroth (1968). The principal objective of this research was to present a unified picture of the stress-strain behaviour of soils, to correlate the results of various types of tests and to obtain the constitutive equations describing the stress-strain behaviour. The final hypothesis emerging from this work was that the (P', q, ω) state paths for all tests would lie within a 'state domain' bounded by two surfaces of limiting states (Rendulic and Hvorslev surfaces). The continuous yielding of a sample was said to be represented by a state path, which would rise to the 'state boundary surface' and remain on that surface until eventually the sample would reach the critical condition, termed the 'critical state', where the sample can continue to distort without further change of e, P' and q.

Following a suggestion by Calladine (1963); Roscoe, Schofield and Thurairajah (1963) assumed that the vertical projection of the elastic swelling curve on to the state boundary surface represented a curve of limiting elastic states and the projection of this curve on to the (P', q) plane was assumed to provide both the yield locus and the plastic potential. This assumption together with the assumption

that distortion is irrecoverable (i.e., $\delta\varepsilon = \delta\varepsilon^P$) and that the elastic volumetric strain is only a function of P' (given by $dv^e = \frac{k}{1+e} \frac{dP}{P'}$; k being the slope of the swelling curve in e-ln P'plot and e being the initial void ratio) , was made use of to predict the shear strains. One further assumption regarding the rate at which the energy is dissipated during shear distortion was made to the effect that $\frac{dw}{d\varepsilon} = MP'$, where W is the energy dissipated within the sample, ε a distortional shear strain parameter, defined such that $d\varepsilon = \frac{2}{3}(d\varepsilon_1 - d\varepsilon_3)$ and M a fundamental soil constant.

The following relations for the Cam clay, have been given based on the assumptions discussed:

Incremental volumetric strain is given by,

$$\delta v = \frac{1}{1+e} \left(\frac{\lambda - k}{M} \delta\eta + \lambda \frac{\delta P}{P'} \right) \tag{2.1}$$

Incremental shear strain is given by,

$$\delta\varepsilon = \frac{\lambda - k}{1+e} \left[\frac{P' \delta\eta + M \delta P}{MP'(M-\eta)} \right] \tag{2.2}$$

Yield locus is given by,

$$\eta = M \log_e \frac{P_o}{P'} \tag{2.3}$$

State boundary surface is given by

$$\eta = \frac{M}{1-k/\lambda} \log_e \frac{P_e}{P'} \tag{2.4}$$

where P'_o is the value of P' when $\eta = 0$ and P_e is analogous to the 'equivalent pressure' proposed by Hvorslev (1937). This model is developed for "wet" clays (normally & lightly overconsolidated clays) only.

The Cam clay model invariably overpredicts the observed values of the strain increments for changes of η, at small values of η, even though excellent agreement was found for changes of η at larger values of η. This model also overpredicts K_o (Roscoe and Burland, 1968). However, the predictions by this model have only been checked for tests on isotropically consolidated samples.

2.3.2.2 Modified Theory after Burland

Burland (1965) improved upon the Cam clay model by adopting a different expression for the dissipated work:

$$\delta W = P' \sqrt{(\delta v^P)^2 + (M\ \delta\varepsilon^P)^2} \tag{2.5}$$

Retaining the other assumptions of the Cam clay, following expressions have been suggested:

$$\delta v = \frac{1}{1+e} \left[(\lambda - k) \frac{2\eta\ d\eta}{M^2 + \eta^2} + \lambda \frac{\delta P}{P}\right] \tag{2.6}$$

$$\delta\varepsilon = \frac{\lambda - k}{1+e} \left(\frac{2\eta}{M^2 - \eta^2} \right) \left[\frac{2\eta \; \delta\eta}{M^2 + \eta^2} + \frac{\delta P}{P} \right] \tag{2.7}$$

yield locus is given by

$$\frac{P}{P_o} = \frac{M^2}{M^2 + \eta^2} \tag{2.8}$$

Equation of the state boundary surface is given by

$$\frac{P}{P_e} = \left(\frac{M^2}{M^2 + \eta^2} \right)^{(1-k/\lambda)} \tag{2.9}$$

While excellent agreement was found between prediction and observation of the state boundary surface, there was a tendency for the underprediction of both shear and volumetric strains. This represented an overall improvement over Cam clay model. Again the predictions from this model have only been checked for tests on isotropically consolidated samples, as shown by Roscoe & Burland (1968). Newland (1973) has shown that this theory considerably overpredicts the shear strains for undrained and q-constant stress path tests conducted on anisotropically consolidated samples. Obviously this requires confirmation by a detailed study.

2.3.2.3. Roscoe and Burland Model

So far it was assumed that no shear distortion of any type can be associated with a state path beneath the state boundary surface. Roscoe and Burland (1968) have given the experimental

evidence to show that "while the concept of an elastic limit line lying on the state boundary surface is a good approximation for volumetric strains, considerable plastic shear distortion does take place for state paths observed beneath and on the state boundary surface". It is observed from experiments that irrecoverable or plastic strains often occur from the very beginning of the deformation process. Moreover, during undrained loading of soil, although the volume changes are zero, considerable irrecoverable shear strains may take place. Roscoe and Burland (1968) suggest the use of an additional yield surface parallel to the P'-axis. The introduction of an additional yield surface of this type permits the retention of the normality rule of plasticity and allows the prediction of plastic shear distortions beneath and on the volumetric yield surface. They suggest the addition of the term $(\varepsilon^p)_{v}P$ (shear distortion corresponding to constant volume test) to the shear distortion predicted by Burland (1965) for each η value.

The comparison of the predictions using this model shows that it gives excellent agreement in the beginning, thereafter Burland (1965) shows better predictions. The volumetric strains are slightly underpredicted. Unfortunately, all the comparisons made from these models, involved tests with isotropic consolidation stress history and these showed very good agreement. However, the results of an excellent series of tests carried out by Lewin and Burland (1970) on anisotropically consolidated samples were never compared with the

predictions by these theories. Newland (1973) has shown that Burland's model considerably overpredicts the shear strains of q-constant and undrained tests carried out on anisotropically consolidated samples. Needless to say, Roscoe and Burland's (1968) model would show still larger predictions for these tests. Though the power behind the generalized theory is never in doubt, it is open to speculation for its capability to accurately predict the insitu behaviour, as the test studies of Newland (1973), Lewin and Burland (1970) have shown. All these studies have either been conducted on Kaolin or on such materials which had low k/λ ratio. Hence, a detailed investigation on anisotropically consolidated natural clay (having high k/λ), encompassing a number of stress paths, is desired to check the suitability of these models to predict the observed behaviour.

2.3.2.4 Prévost and Höeg Model

The Cam clay, Burland and Roscoe and Burland models as described above were developed at Cambridge. These models have been given for "wet clays" only.

Prévost and Höeg (1975,a) have recently incorporated Roscoe and Burland's suggestion of a horizontal yield surface in addition to the usual yield surface, to take into account the additional component of plastic shear distortion for stress paths inside and on the volumetric yield locus.

In another paper, Prévost & Høeg (1975) have extended the Cambridge models to take into account the strain softening behaviour in its formulation. This is an important contribution. The predictions from these models have yet to be assessed.

Thus it can be seen that the incremental theory of plasticity is a powerful tool and can handle the dilatant, stress path dependent soil behaviour, as opposed to the theory of elasticity approach, which cannot take these factors into account. If should, however, be mentioned that the use of associated flow rule of plasticity would lead to erroneous predictions of shear strains, as the direction of the plastic strain increment vectors is dependent on the direction of the stress path (LeLievre and Wang, 1971; Lewin and Burland, 1970). A more correct approach would appear to be on the lines suggested by Lade and Duncan's (1975), who use a non-associated flow rule.

2.3.3 Semi-Empirical Methods

2.3.3.1 Roscoe and Poorooshasb Model

According to Roscoe and Poorooshasb (1963), the incremental strain associated with a given stress increment can be considered as the sum of two components that occur in a constant volume process (undrained test) and a process in which the stress ratio remains constant (anisotropic consolidation test). The relationship was expressed in the form:

$$\delta\varepsilon_1 = \left(\frac{\partial\varepsilon_1}{\partial\eta}\right)_v \delta\eta + \left(\frac{\partial\varepsilon_1}{\partial v}\right)_\eta \delta v \tag{2.10}$$

where $\delta\varepsilon_1$ is the total axial strain increment and $\delta v = (\delta\varepsilon_1 + 2\delta\varepsilon_3)$ is the total volumetric strain increment.

The first component $(\frac{\partial\varepsilon_1}{\partial\eta})_v$ is obtained from an undrained test, whereas $(\frac{\partial\varepsilon_1}{\partial v})_\eta$ is obtained from anisotropic consolidation tests.

Poorooshasb and Roscoe (1963) indicated the graphical approach, which can be conveniently used to predict the strains. Predicted and experimental values are compared for conventional drained tests. The predictions were quite good.

This is the first systematic semi-empirical approach used to predict the strains for wet clays. However, this method has not been tried to predict the strains for other stress paths.

2.3.3.2 Wroth and Loudon Approach

Wroth and Loudon (1967) have shown that the results of a series of undrained tests on samples of Kaolin with varying degrees of overconsolidation, when plotted non-dimensionally in P/P_e and q/P_e space, reveal a family of distinct contours of equal increment of strain, parallel to the P/P_e axis in the light overconsolidation range. With the help of these contours, it is shown that the complete stress-strain curve of the conventional drained test

starting from a lightly overconsolidated state can be satisfactorily predicted. The prediction of only one test is shown. However, the parallelism of the contours obtained by Wroth and Loudon for isotropically consolidated samples, will not hold good for k_o-consolidated samples, as has been shown by Parry and Nadarajah (1974).

2.3.3.3 Wroth's Model

Wroth (1968) suggested that the results of a wide variety of tests on normally consolidated clays may be predicted from the results of two series of tests (i) anisotropic consolidation tests (η = constant) and (ii) P-constant tests ($P' = \frac{\sigma_1' + 2\sigma_3'}{3}$ = constant) as follows:

$$dv = \frac{\partial v}{\partial P}\, dP + \frac{\partial v}{\partial \eta}\, d\eta \qquad (2.11)$$

$$d\varepsilon = \frac{\partial \varepsilon}{\partial P}\, dP + \frac{\partial \varepsilon}{\partial \eta}\, d\eta \qquad (2.12)$$

The factors $\frac{\partial v}{\partial P}$ and $\frac{\partial \varepsilon}{\partial P}$ are obtained from the anisotropic consolidation tests, whereas $\frac{\partial v}{\partial \eta}$ and $\frac{\partial \varepsilon}{\partial \eta}$ are obtained from P-constant tests. Wroth has suggested that Eqs. 2.11 and 2.12 are general and can be used to predict the stress-strain behaviour along various stress paths. These parameters have been evaluated from P-constant loading & unloading tests and anisotropic consolidation and swelling tests. The stress level dependency has been taken into consideration. The prediction of conventional

drained, special drained test with σ'_3 reducing, q-constant test, undrained test, anisotropic consolidation test, K_o, and k_o-unloading, have been given using this approach.

Wroth's model appears to be a powerful one, which has the ability to predict both loading and unloading stress paths. The predictions of the results of a few stress path tests conducted on isotropically consolidated samples have been made by Wroth. However, Wroth's model should be checked for its predictive power, with the test results (different stress paths) conducted on k_o-consolidated samples.

2.3.3.4 Newland's Approach

Newland (1973) method is based on the premise that any loading stress path (wet clay) may be considered to be a combination of the two phases, namely, undrained shear (with q increasing) and consolidation with q-constant. Based on this model, he predicts the results of anisotropic consolidation and the conventional drained tests.

The predictions show good agreement with the experimental results. Newland's results (q-constant) conducted on anisotropically consolidated Keolin could not be predicted by the Cambridge models.

2.3.3.5 Lewin's Approach

Lewin (1971) conducted a no. of anisotropic consolidation tests both on the compression and the extension side on Llyn Brianne Slate dust

(same material as used in the investigation reported in Lewin and Burland, 1970). Experimental evidence is presented which suggests that where a saturated clay is subjected to anisotropic consolidation at constant stress ratio, the direction of the strain increment vector may be represented by a plastic potential which forms a surface of revolution about the stress space diagonal. From this it follows that the three-dimensional strain behaviour for anisotropic consolidation can be established from a flow rule which may be determined from simple axisymmetric triaxial tests. It is shown that this flow rule lies close to a straight-line relationship which is defined by a single parameter ϕ'

$$\alpha/\beta = \tan^{-1}\left[\frac{2\sqrt{2}\sin\phi'}{3-\sin\phi'}\right]/\pi/2 \qquad (2.13)$$

where ϕ' is the angle of shearing resistance, α is the angle subtended by the anisotropic stress path in $(\sigma_1' \; \sigma_2' \; \sigma_3')$ stress space with respect to the stress space diagonal, and β is the angle, similarly defined, relating to the strain increment vector. It is then shown that the three-dimensional deformation behaviour during anisotropic consolidation can be represented by contours of equal strain increment ratio drawn on the octahedral plane. It is also shown that plane strain will result from any one of a specific family of anisotropic consolidation stress paths and this family forms part of a conic surface in stress space. These conclusions apply to soils which behave isotropically under an increasing isotropic stress.

In a later paper, Lewin (1973) considerably developed these concepts by conducting constant stress ratio tests where the value of the stress ratio was changed to a new value during the course of the test, the stresses being then further increased at that new value of stress ratio. The particular concern was with the direction of the strain increment vector. The results show clearly that stress history has a considerable effect on the direction of the strain increment vector and this effect is very persistent even with a substantial rise in the stress level. The observed plastic potentials for three stress histories have been compared and they appear to exhibit some form of 'kinematic' behaviour because it appears to translate with the stress state. Finally this paper offers two possible methods of interpreting the results, which may be used in the prediction of stress-strain behaviour. The first method suggests that the plastic potential is elliptical in shape and symmetrical, possibly even in σ'_1, σ'_2, σ'_3 space, about an axis that lies between the stress space diagonal and the original stress path. The second method suggests that there is a simple relationship between the direction of the new stress path α and the direction of the strain increment vector β.

Lewin (1975) suggests an extension of Roscoe and Poorooshasb method. In the first part of the paper, deformations are predicted for normally consolidated saturated clay tested in conventional triaxial apparatus. The effects of stress history and anisotropy on

the above lines are examined and the Roscoe-Poorooshasb's analytical model is modified to take these factors into account. It is also suggested that the three curves used in the Roscoe - Poorooshasb model, viz. undrained stress path, plastic potential and stress-strain curve for an undrained test can be related. It is then suggested that the Roscoe-Poorooshasb method can be extended to deal with stress paths in which the three principal stresses can vary independently. Experiments carried out by Hambly (1972) in plane strain and Wood (1973) in true triaxial apparatus have been used to test the proposed model under three-dimensional stress conditions and the results compare quite favourably with prediction. According to Lewin, his method represents a <u>useful advance</u> on the Roscoe-Burland model.

It is thus clear, that the semi-empirical approach as discussed, may provide an alternative and sound approach, at least at the present time (1975). Out of the models discussed, Wroth's (1968) and Roscoe and Poorooshab's (1963) models appear to be better suited for adaptation. Clearly Wroth's model is more versatile as it can be used for predicting various stress paths, as compared to the other model, which is limited on the wet clay side.

2.4 TIME EFFECTS - CREEP

Time effect are extremely important in the study of general stress-strain-time behaviour, as it will give an idea of the gain/loss of strength with time, depending upon the field problem. When loads

are applied to the clay strata, initially due to undrained conditions (drainage takes place slowly only after the dissipation of pore pressure), soil exhibits creep. Later on, after the pore-pressures are dissipated, drained creep takes over. Depending upon the field conditions of drainage and layer thickness, undrained creep may continue for quite sometime. Undrained creep has been the subject of study by many workers, and a detailed review has been given by Edgers et. al. (1973).

2.4.1 Drained Creep Behaviour

2.4.1.1 Drained Creep under k_o-Condition

Drained creep has been investigated by many workers, mainly from Oedometer tests. Mesri (1973), in an excellent review, has discussed the effects of many factors on the coefficient of secondary creep or compression (c_α). Drained creep along k_o or different anisotropic stress ratios in triaxial has been studied by Barden (1969), Newland (1973), Ladd and Preston (1965), Yamanouchi and Yasuhara (1975).

The main conclusions arising out of these studies are: (i) For small stress ratios upto K_o, C_α virtually remains constant beyond which it increases (Ladd & Preston, 1965) (ii) the variation of C_α with η, at higher η values, is linear both corresponding to $C_{\alpha v}$ and $C_{\alpha\varepsilon}$ (Yamanouchi and Yasuhara, 1975); nonlinear for $C_{\alpha\varepsilon}$, (Newland, 1973). (iii) creep rate (C_α) as obtained from a triaxial test is generally higher than that obtained from an oedometer, due to side friction in the oedometer.

These tests are valid only when k_o-condition exists in the field. However, generally, insitu conditions are non-k_o and hence the rates in the field are likely to be higher than the rates determined from an Oedometer test or a k_o-triaxial test.

2.4.1.2 Drained Creep under Non-k_o Condition

Very little work has been done on this important aspect. Drained triaxial and direct-simple shear creep tests on clay soils have been reported by Vialov and Skibitsky (1961), Murayama and Shibata (1961, 1964), Barden (1969), Walker (1969), Bishop and Lovenbury (1969), Newland (1973), Poulos et. al. (1975), Yamanouchi and Yasuhara (1975). A few important contributions will be discussed. Murayama and Shibata (1961, '64) have performed drained triaxial compression tests on isotropically consolidated samples of Osaka clay. Test results indicated a linear variation of strain with log time after the initial pore pressures had dissipated. Creep tests on two stress paths were conducted (i) the conventional drained test (ii) q-constant test. For q-constant test, $C_{\alpha\varepsilon}$ was found to remain constant with q, whereas for the conventional drained test, $C_{\alpha\varepsilon}$ increases linearly with stress level upto a "yield value", after which it increases very rapidly with stress level. At stresses higher than the yield value, the drained creep developed into creep rupture. This is one of the early investigations which showed the effect of stress path on creep rates.

Barden's (1969) study was concerned mainly with the conventional drained test. The logarithmic axial creep rate ($C_{\alpha\varepsilon_1}$) was found to be independent of pressure increment ratio, height of the sample, mean effective stress and direction of previous loadings. Barden found that the stress ratio was the only factor which affected creep rates most. $C_{\alpha\varepsilon_1}$ was found to increase approximately linearly with the stress ratio. Barden mentions that the creep rates may be influenced by stress path.

Walker (1969) conducted both triaxial and simple shear tests. The volumetric creep rate, was found to be independent of stress ratio, whereas the shear creep rates increased linearly with the stress ratio, agreeing with the findings of Murayama and Shibata and Barden. However, $C_{\alpha v}$ can't retain the same value throughout, because as failure approaches, the sample would flow at a very small volumetric creep rate. Thus $C_{\alpha v}$ should decrease after reaching a certain value. Infact Walker's experimental points, show a tendency in that direction.

Bishop and Lovenbury (1969) tests on the overconsolidated London clay and a normally consolidated Pancone clay from Pisa are probably the longest creep tests available. The observed behaviour differs considerably from the behaviour described in earlier studies. A simple continuous variation between axial strain and log time was observed for only 60 $\pm$ 20 days for the Pancone clay and 200 $\pm$ 100 days

for the London clay.

The "instabilities" which later developed suggested a fundamental modification in soil structure to Bishop and Lovenbury. However, inspite of the very specialized equipment especially developed for this purpose, it can be said that such instabilities could result from temperature changes or external vibration or due to physicochemical changes in the sample.

Edgers et. al. (1973) point out that if instabilities are considered as a fundamental phenomenon, it will render the many simple logarithmic or power laws used to represent the time-dependent behaviour of creeping soils.

The unloading tests (with σ_3' decreasing) indicate slightly smaller axial strain rates than the conventional drained test.

Newland (1973) has conducted q-constant tests mainly and have shown that the shear creep rate for this stress path is independent of the stress ratio. This agrees with the findings of Murayama and Shibata (1964).

But Newland suggests a constant decrease in volumetric creep rates with increase in η . This does not agree with the findings of Walker.

Yamanouchi and Yasuhara (1975) have described the results of a number of P-constant drained creep tests and show a linear increase both in axial and volumetric creep rates with the stress ratio η .

However, the volumetric creep rate variation is not compatible with the known fact that normally consolidated clays at failure starts flowing, at small constant volumetric creep rate.

It is clear from the available data on drained creep rates, that, as of today (1975), there is a lot of confusion in the variation of the volumetric creep rates with stress ratio. Also, only Bishop and Lovenbury and Murayama and Shibata give creep rates for two stress paths on the same soil. Hence, there is an apparent need of conducting drained creep tests on a given soil, along different stress paths.

2.4.2 Rate Process Theory

The theory of absolute reaction rates, commonly known as rate process theory, is one of the most commonly used models for describing the deformations of soils. The development of the theory assumes that flow units (atoms; molecules or particles) are constrained from movement by energy barriers which separate adjacent equilibrium positions. Upon the application of an external load or stress, sufficient activation energy is supplied to these flow units to surmount the energy barriers.

The three parameter equation finally suggested by Singh and Mitchell (1969) is:

$$\dot{\varepsilon} = A\, e^{\bar{\alpha}\bar{D}} \left(\frac{t_1}{t}\right)^m \qquad (2.14)$$

where $\dot{\varepsilon}$ = strain rate

ε = strain (axial, volumetric, shear)

$A = \dot{\varepsilon}$ at $t = t_1$ and $D = 0$ (extrapolated value)

α = slope of linear portion of ln $\dot{\varepsilon}$ vs deviator stress D plot

D = deviator stress, $\sigma_1 - \sigma_3$

D_{max} = deviator stress causing failure at conventional strain rate, $(\sigma_1 - \sigma_3)$

$\bar{D} = D/D_{max}$

$\bar{\alpha} = \alpha D_{max}$.

t = time after application of deviator stress

t_1 = reference time

m = creep coefficient = absolute value of slope of (assumed) straight line portion of log $\dot{\varepsilon}$ vs log t plot.

Singh & Mitchell (1968) place great emphasis on the creep potential, 'm', which they consider to be a material property only slightly influenced by test conditions and the stress level. The parameter '$\bar{\alpha}$' indicates the stress intensity effect on creep rate. The 'A' parameter, although it does not represent an actual physical quantity, reflects the order of magnitude of the creep rate. It is in effect, a soil property reflecting the composition, structure and stress history of the soil.

Singh and Mitchell state that numerous tests indicate that 'm' is a material property only slightly influenced by test conditions and

the stress level. Edgers et. al. (1973) feel "that there are insufficient data for a variety of test conditions to justify this conclusion. It is possible that 'm' may be affected by stress history, stress system prior to shear, the degree of aging prior to shear, and the stress system applied during shear".

The Berkeley model has been suggested to predict both undrained and drained creep behaviour. For undrained creep rates, the model has been used by many investigators (see Edgers et. al., 1973 for details). However, this model has not been used to predict the drained creep rates so far. There is thus great need to evaluate the three parameters from the results of drained test to check the universality of predictions as claimed by Singh and Mitchell. Also the dependence of these parameters on stress path should be assessed.

2.4.3 Rheological Models

The phenomenological approach is often used for quantitatively describing the behaviour of most engineering materials. A large number of composite rheological models have been reported in the soil mechanics literature. Refer Edgers et. al.(1973)for details. It appears from the work of many investigators, that although these models are useful analogies for describing the mechanics of soil deformations, they are both too complex and too approximate and limited for quantitative prediction of the actual stress-strain-time behaviour of many soils (Edgers, et. al., 1973).

2.5 SCOPE OF INVESTIGATION

The preceding discussions bring out clearly the background and the trend of development of research in the study of stress-strain-time behaviour of clays. In the light of the state of understanding on the behaviour of clays, the following aspects are considered in the present thesis.

(a) Stress-strain behaviour of anisotropically consolidated clay along various stress path

(b) Drained creep behaviour of anisotropically consolidated clay along various stress paths

(c) Prediction of the observed stress-strain-time behaviour by [illegible] a [illegible] model and comparison of the test results from predictions by other available models.

CHAPTER 3

EXPERIMENTAL SETUP AND TEST DETAILS

3.1 GENERAL

The laboratory tests on undisturbed samples of soils are highly desirable to predict the insitu behaviour. But as the undisturbed samples are generally found to be non-uniform, and differ from each other, their use in a fundamental laboratory study is limited. Moreover, it is quite difficult to obtain good undisturbed samples. On the other hand, as the laboratory prepared resedimented soil samples are very uniform and similar to each other, the results of tests conducted on such samples can be usefully compared with each other to clearly define the observed behaviour. Moreover, any stress-strain theory which is developed (this appears to be the trend in the seventies) requires for its validity, tests on uniform soil samples and it is only when the theory is able to correctly predict the behaviour under the most ideal laboratory conditions, it can be considered as appropriate and then thought could be given for its application to predict the complicated field behaviour. In this investigation, a soft Indian marine clay was used. Its properties along with the equipment & testing details are now described.

3.2 MATERIAL USED FOR THE STUDY

The remoulded clay used in this study is Rann of Kutch clay procured from 3 inch dia tube samples from little Rann of Kutch area in Gujarat. The Rann of Kutch was a shallow arm of sea during pleistocene and now it is a flat depression inundated by the flood waters of many small rivers draining into that area during monsoon and by tidal waters during other seasons. The deposit in its natural state is normally consolidated. This marine clay is soft and contains 12 percent organic matter in its structure. Its other properties are: LL = 91%; PI = 49%; clay fraction = 84%; specific gravity = 2.71; activity = 0.58; $k/\lambda = 0.4$. It contains illite, quartz, kaolin and calcite.

3.3 SAMPLE PREPARATION

The clay was thoroughly mixed with deaired distilled water and made into a thin slurry having a water content of about two and half times the liquid limit. This was stored in a glass container for tests. The samples from this slurry were prepared by sedimentation. Eight samples could be prepared at one time using the sampling bench which was fabricated at IIT Kanpur workshop. See photograph (Fig. 3.1). $1\frac{1}{2}$ inch dia perspex tubes about 10 inch long were mounted erect, through 'O' rings, on stainless steel pedestals having an outlet as shown. A porous stone along with a filter paper

was kept over the pedestal. The slurry was poured into the tubes slowly, driving out air by stirring the slurry and slightly tilting the tube along with the base. The slurry was allowed to consolidate under its own weight for a day. Porous stone and filter paper were then put at the top and left for a day. After this, the hanger was put above the porous stone and the sample was allowed to consolidate for a day. The loads were then put in succession in small increments, each load being retained for a day. The loading was stopped when the sample finally was consolidated under 3 psi pressure. Samples, so formed, were then pushed out slowly from the tube and cut in 3 inch lengths by means of a thin wire saw and then mounted on the triaxial base as detailed below.

3.4 TEST EQUIPMENT USED

Fig. 3.2 shows the photograph giving details of the major part of the experimental setup used.

3.4.1 Triaxial Cells

Norwegian type triaxial cells were fabricated here. The frictionless piston assembly with rotating type bushing procured from the Norwegian geotechnical institute were fitted to the cells. The cells have a pedestal of 1.5 inch diameter and have two outlets at the bottom for volume change and pore pressure measurements.

3.4.2 Volume Gauges

To measure small changes in volume with accuracy, u-shaped volume gauges were fabricated here from uniform thin bore, thick walled glass capillary tubing. The ends were tightly plugged with hollow tapered needles fitted into the glass ends with araldite. Glass tubes of various bores were used to enable the measurement of volume change to the accuracy of 0.0028 cc to 0.008 cc. Entrapped air bubble was used as an indicator in the volume gauge. The volume gauges were mounted on wooden boards with cm scale fitted and were kept horizontal during the tests. The back pressure was applied to the samples through the burette and was maintained throughout the test.

3.4.3 Self Compensating Mercury Control System

This system was used to apply and maintain the cell pressure and the back pressure to the sample throughout the duration of the test as described in Bishop & Henkel (1962).

3.5 SAMPLE MOUNTING DETAILS

The triaxial base was connected to a burette (50 cc) and boiling distilled water was flushed through the pedestal and the bottom tubings of the base to drive out the air from the system. A thin rubber tube coated with grease was used to enclose the pedestal for housing the porous stone and also to

prevent the membrane enclosing the sample from getting punctured at the pedestal base. Boiled porous stone was kept over the pedestal and the boiling water was allowed to flow through the porous stone. Six filter strips each 1/4 inch wide were tucked in the housing between the porous stone and the rubber tube. The soil sample cut into 3 inch length by means of a wire-saw was slowly set in place over the pedestal. Filter paper discs (Whatman no. 1) were used both at the bottom and at the top of the sample. Care was taken to wipe off any excess water appearing over the bottom filter paper before transferring the sample over the pedestal. Filter strips were put in position around the sample and a perspex cylindrical cap having a hole in the centre along with a stainless steel ball to house the piston was kept along with a filter paper at the top. Two thin rubber membranes soaked earlier in water were then slipped over the sample carefully one by one. Three to four 'O' rings were used at either end to seal the membranes to the sample at the ends. Care was taken to drive out the air in between the sample and the membrane. The triaxial cell cover was then put and tightened to the base with nuts. The cell was filled with deaired distilled water. The top few inches were filled with transformer oil till it started coming out of the air release valve at the top which was then immediately plugged. The piston was then set in position and clamped at the top. The burette connecting the cell bottom tubing was then

lowered so that any water remaining in between the sample and the base including the rubber tube gets driven out into the burette. The cell pressure is then applied.

3.6 BACK PRESSURE SATURATION AND ANISOTROPIC CONSOLIDATION

After the sample was mounted as described, it was subjected to an alround pressure of 5 psi with the help of self compensating mercury control and was allowed to consolidate for a day. Volume measurements were made with the help of 50 c.c. burette.

The 50 c.c. burette was then disconnected and instead the ends of the cell bottom were connected with the U-tube burette having an entrapped air bubble taking care that air does not enter the connection. The cell pressure was slowly increased to 35 psi and side by side a back pressure of 30 psi was slowly builtup and applied to the sample via the burette through another set of mercury control system. The sample was then left under back pressure of 30 psi for a day. One day period was found to be sufficient for complete saturation for this clay (Varadarajan, 1973). The light aluminium hanger was then put over the piston & cell assembly so that this just balances the upthrust on the piston due to the applied cell pressure of 35 psi. Dial gauge was mounted on the top of the hanger to record the vertical strains(See Fig. 3.2).

To reach a desired anisotropic ratio from the isotropic stage, a one stage P-constant test was conducted by reducing the alround pressure by a calculated amount and by increasing the vertical pressure by double the amount so that the mean pressure ($p' = \frac{\sigma_1' + 2\sigma_3'}{3}$) stays constant at 5 psi. This change was maintained for a day.

Having reached a desired basic stress state ($q/p' = 0.1$; 0.2; 0.4) through P-constant test, anisotropic consolidation was carried out in three steps along each anisotropic line so that a mean pressure of about 23.3 psi was reached. This was carried out by increasing the cell pressure σ_3' by a known amount via the constant pressure system and side by side, increasing the vertical pressure σ_1' by a precalculated amount by putting additional weights on the hanger, so as to maintain the desired stress ratio (q/p'). One day was allowed for each increment along the anisotropic consolidation line. Having reached the desired pressure of 23.3 psi, which is the starting pressure for all subsequent drained tests, the sample was allowed another 16 to 24 hour time before the start of shearing. Two basic stress states corresponding to $q/p'=0.2$ and 0.4 have mainly been used. Only one test was conducted from the basic stress state of $q/p' = 0.1$.

3.6.1 Unloading to Produce Lightly Overconsolidated Samples.

A few tests were conducted from a lightly over-consolidated state. An OCR of 1.25 was achieved by decreasing both σ_3' & σ_1' such that the ratio $q/p' = 0.4$ was maintained.

3.7 DETAILS OF DRAINED TEST.

From each of the two basic stress states (q/p'=0.2 and 0.4), where the starting point corresponds to a mean pressure of 23.3 psi, the samples were subjected to various stress paths (probes) in various directions covering almost the entire stress domain. The various stress paths numbered A,B,C...etc. are shown in Figs. 3.3(a) and (b). For these probes, vertical strains are indicated by the dial gauge and the changes in volume by the U-tube burettes. Small changes in the alround pressure over and above the back pressure were measured by means of a differential mercury manometer.

3.7.1 Definition and Magnitude of Stress Probe

As the magnitude of the probe is an important factor in stress-strain measurements and in order to be able to correlate the strains from various tests, the following definition was adopted [Lewin and Burland (1970)] . The basic stress state from each anisotropic line was defined as $\sqrt{\sigma_1'^2 + 2\sigma_3'^2}$ in $(\sigma_1' - \sigma_3')$ space. This represents the length OI in Fig. 3.3(a). The magnitude of the stress

probe $\sqrt{(\Delta\sigma_1')^2 + 2(\Delta\sigma_3')^2}$ was decided as 20 percent of the basic stress state i.e., $0.2\sqrt{\sigma_1'^2 + 2\sigma_3'^2}$. In order to obtain a stress-strain curve for each probe, the total magnitude of 20 percent was applied in four equal increments of 5 percent each i.e. $0.05\sqrt{\sigma_1'^2 + 2\sigma_3'^2}$, each step being allowed for a day. Thus four days were required to completely apply the full magnitude of each probe. The probes are shown in fig. 3.3(a) marked IA, IB,...etc. The same probes in q-p' space are shown in Fig. 3.3(b).

3.7.2 Unloading along Various Probes

In order to observe the behaviour during unloading along each of these probes, unloading was done in four equal steps along each stress probe.

3.7.3 Special Tests

Two special tests (numbered I & II) have been conducted from the basic stress state of q/p' = 0.4. A number of stress paths have been incorporated in each.

Another test was conducted where the sample after being consolidated anisotropically along q/p' = 0.4, was left for 20 days for delayed consolidation to take place. It was then sheared along the conventional (A) stress path. This test was performed to simulate the actual field behaviour, as the soft marine clays insitu

are generally found to possess a light degree of overconsolidation because of delayed consolidation.

In order to have an idea of the preconsolidation pressure (P_c) developed during the delayed consolidation, a consolidation test was conducted to determine P_c-value for the soil.

None of the tests were conducted upto failure except for the one conducted from $q/p' = 0.1$ stress state.

3.8 TIME READINGS AND TEST CONDITIONS

To have an idea about the drained creep behaviour of the marine clay along various stress paths, dial gauge readings and the bubble movement in the burette were observed at different time intervals for each stage of the probe. Inspite of all the care, measurements of volume changes during drained creep, are likely to be less accurate than the corresponding axial strains, since considering the small magnitude of the volume changes involved, such factors as leakage past the membranes, 'O' rings etc. may be significant for these tests extending over fifteen days. Poulos's (1964) detailed investigation on the control of leakage in the triaxial test shows that for the equipment used by the author in this investigation, the maximum expected leakage rate could be taken as 0.00046 percent per day (although this figure is on the higher side).

All tests were conducted in an airconditioned room where the temperature fluctuations were not much, barring a few sudden power break downs, which were unavoidable. No correction for filter paper drains and rubber membranes was made to the measured values. No appreciable non-uniform deformation of the samples was observed due to the relatively small probes used, apparently showing that the end restraint effect is almost negligible. It is felt that as all the tests were conducted under exactly similar conditions and as this is a comparative study, the behaviour along various stress paths to be presented in the subsequent chapters, at least qualitatively, is likely to remain unaffected, although quantitatively, the results may be affected to some extent by the aforesaid factors.

Water content determination was generally done both at the start and at the end of the test. The size of the sample during consolidation and shearing was corrected by assuming the cylindrical shape of the samples throughout the test.

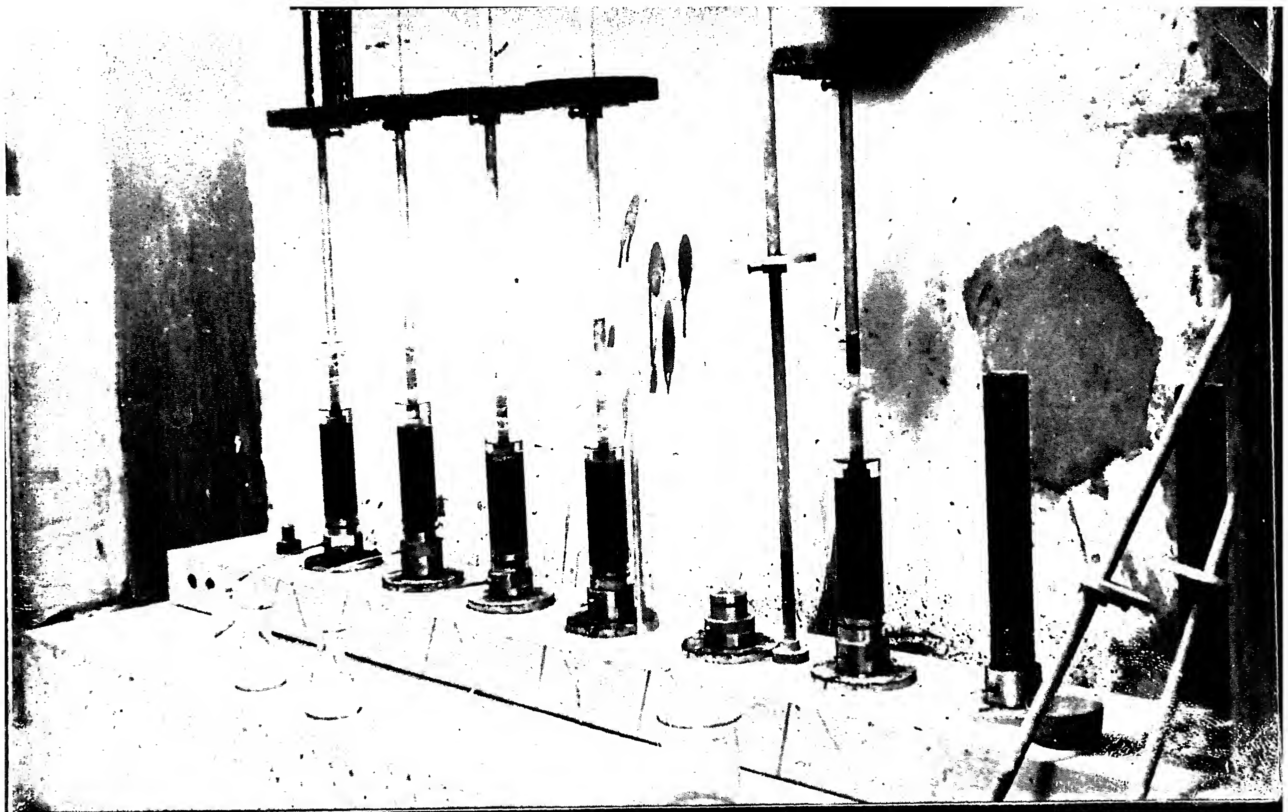


FIG. 3.1 SAMPLING BENCH SETUP

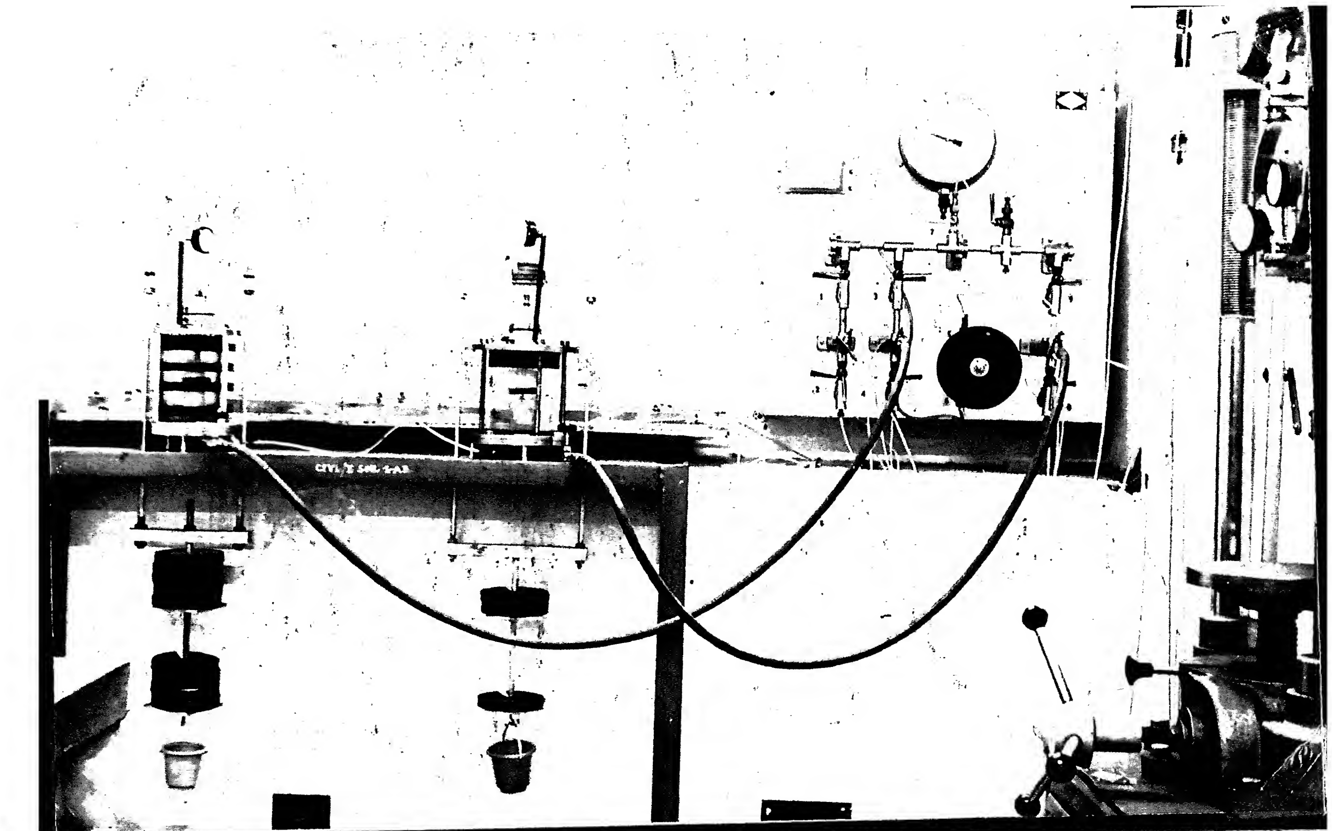


FIG.3.2 DETAILS OF THE EXPERIMENTAL SETUP

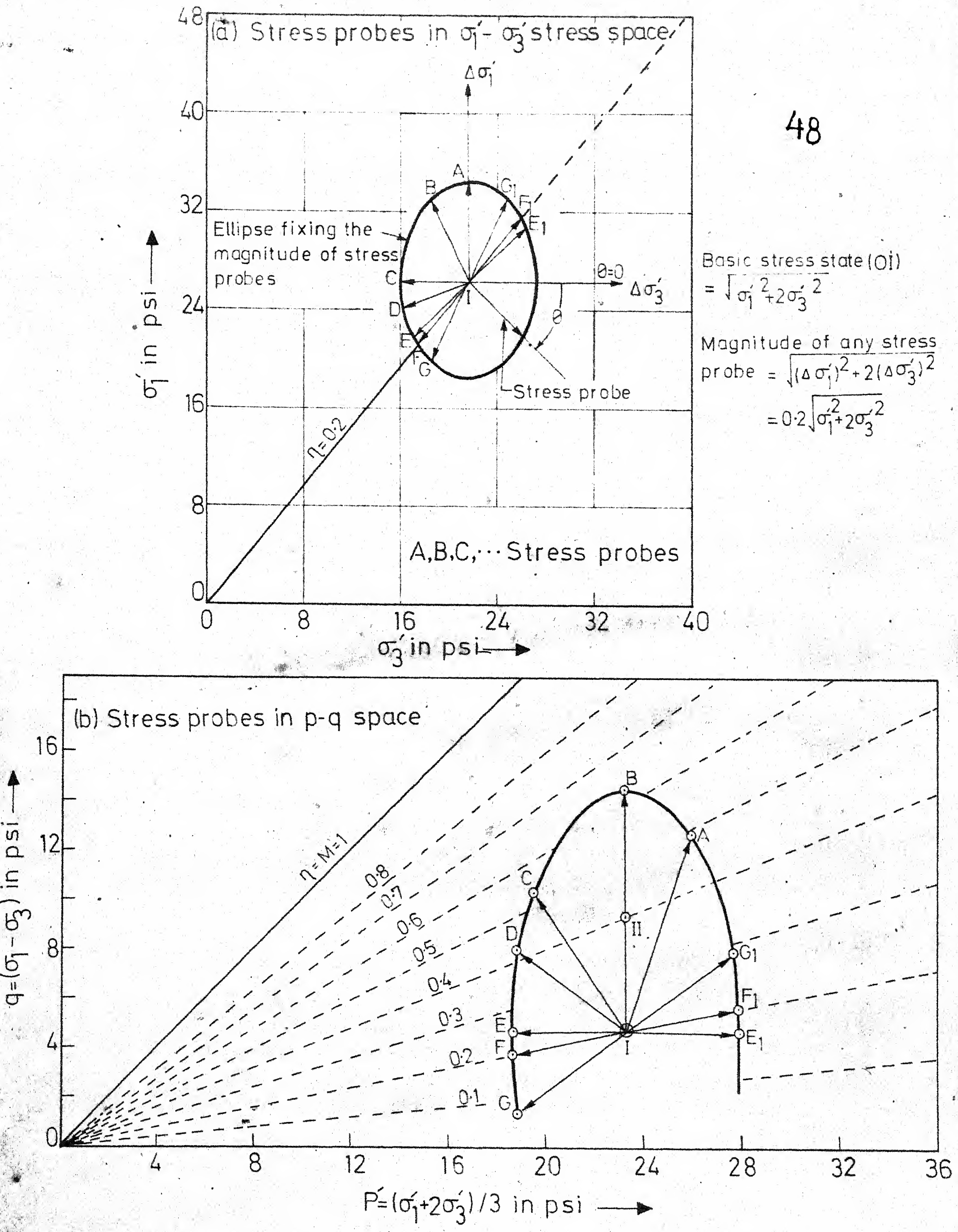


FIG.3·3 DEFINITION & DETERMINATION OF STRESS PROBES IN STRESS SPACE

CHAPTER 4

STRESS-STRAIN BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED CLAY

4.1. GENERAL

Drained triaxial tests have been conducted along various stress probes from two basic stress states $q/p' = 0.2$ and 0.4. The magnitude of the total probe was taken as twenty percent of the distance of the basic stress state from the origin. Four equal increments of five percent each were applied to the sample. Each increment was kept for one day. The sample was then unloaded back along the same probe. Three tests along different stress paths from a lightly overconsolidated state were also conducted. The results of these tests along with two special tests and a complete conventional drained test from the basic stress state of $q/p' = 0.1$ have been presented in this chapter. Stress-strain-volume change behaviours both during loading and unloading for these tests have been given. Various moduli have been evaluated using the theory of elasticity and the effect of stress path on modulus is discussed in detail.

4.2 ISOTROPIC AND ANISOTROPIC CONSOLIDATION

One isotropic consolidation test, a number of anisotropic consolidation tests along two basic stress states ($q/p' = 0.2$ & 0.4) and a test along the basic stress state of $q/p' = 0.1$ were conducted on resedimented samples of the clay. The slurry used for preparing

the samples had a water content of 230 percent (about two and half times the liquid limit). Fig. 4.1 shows the virgin consolidation and the swelling curves for isotropic consolidation test. The consolidation and swelling indices are 0.339 and 0.103 respectively, giving k/λ ratio of 0.304. Dotted lines in the figure show the results of test conducted by Varadarajan (1973) on the same clay prepared from a slurry having water content of about 150 percent. λ & k values obtained by Varadarajan are 0.271 and 0.115 respectively ($k/\lambda = 0.425$). The difference in λ-values is about twenty five percent for the two cases. Variation in k is small. This variation in λ-values is in order since the dependence of compressibility of a clay on its natural water content is well known. Lewin and Burland (1970) quote some variation in λ due to the difference in slurry used. Average values of λ and k ($\lambda = 0.3$ and $k = 0.1$) are used in the predictions by various models as discussed in chapter 6.

Fig. 4.2 shows the anisotropic consolidation test results in the form of ω- log p plot. The isotropic consolidation and swelling lines from Fig. 4.1 have also been included in the figure for comparison. The failure line corresponding to $\eta = M = 1$, has been drawn through the points obtained from various tests by Varadarajan on isotropically consolidated samples. The result of the conventional drained test from the basic stress state of $q/p' = 0.1$ is also shown. The failure line as shown, appears to be enclosing most of the failure data available for this clay (except

some points from C & D stress paths).

Points through which $\eta = 0.2$ & 0.4 lines have been drawn, correspond to an average of atleast fifteen tests. $\eta = 0.1$ line has not been shown as only one test was available from this basic stress state. The slope of the failure line is approximately sixteen percent flatter than the slope of the virgin compression line. Slopes of other lines are in between and have been shown in the figure.

This is in contrast to the approximated parallelism of these lines as has been reported by many workers, for example, Henkel & Sowa (1963), Roscoe and Poorooshasb (1963), LeLievre (1967), Lewin & Burland (1970). The variation in λ values with η as observed by them was not much and amounted to a scatter & as such the assumption that λ is independent of η was made. In a recent study by Newland (1973) on kaolin, shown in Fig. 4.3(a), it is clearly observed that the slopes of these lines are different. There is a difference of about eleven percent in the slopes of $\eta = 0.78$ line and the isotropic line. Newland shows a difference of 15 percent between the slopes of $\eta = 0.9$ line and the isotropic line. M is 1.1 for the soil used by Newland and as no failure data is available, the maximum variation in the slopes of isotropic & failure lines has not been given. Newland's work is in agreement with the present findings.

Fig. 4.3(b) shows the directions of the total increment vectors ($\dot{\varepsilon}/\dot{v}$) along each anisotropic line. Although there is a

slight variation in the slopes of the vectors along an anisotropic line, essentially the slope stays constant & the vectors are parallel. This broadly corroborates the findings of Roscoe and Poorooshasb (1963), LeLievre (1967), Lewin & Burland (1970). A detailed discussion on anisotropic consolidation tests will follow in Chapter 6.

4.3 P_c-VALUE DETERMINATION FROM OEDOMETER TESTS

To get an idea of the reserve resistance which the clay can develop, if allowed to undergo delayed consolidation and to correlate the behaviour of the lightly overconsolidated samples with the field behaviour, a consolidation test was carried out. Fig. 4.4 shows the results of this test. The sample was subjected to usual increments of load upto 'd', which corresponds to a pressure of $2kg/cm^2$ (the effective major principal stress at the start of shearing). It was then allowed to undergo delayed consolidation for a period of twenty days. Pressure was then increased in small steps till the expected P_c-value was reached. Regular consolidation increments were then resumed from 'f'. Casagrande construction was done to obtain the P_c-value. Details are given on an enlarged scale in the inset diagram. P_c/P_o for this clay is 1.3 for the twenty day rest period. This is in line with Bjerrum's (1967,1972) findings and suggests that this clay will be able to withstand an additional pressure $\Delta P = (P_c - P_o)$, without undergoing much settlement for pressures upto P_c. Soft marine clays in field undergo secondary consolidation for

thousands of years and develop a reserve resistance. Due to this, the normally consolidated soft marine clays behave as lightly overconsolidated, thus indicating that the insitu stress-strain behaviour can be altogether different than what would be obtained from tests on normally consolidated samples of the clay. This aspect is described later in this chapter.

4.4 EFFECT OF THE MAGNITUDE OF THE PROBE ON STRAIN RESPONSE

As mentioned in chapter 3, the magnitude of the probes in this investigation was assumed as twenty percent of the distance of the basic stress state from the origin in order to have the same length of the probe in the stress space. Figs. 4.5(a) & (b) show the results of F stress path tests from the two basic stress states. Stress path F is the unloading path along the anisotropic line. 'A' is the starting point and ABCDE shows the response obtained from the tests where the full magnitude of the probe is given in four days. AF is the response obtained by giving the full probe in one step and then recording the strains after four days. The conventional isotropic swelling curve is drawn through A (AG) for comparison. The slopes indicated by k have been listed against each line. The maximum variation in k is about nine percent. The points B, C and D in Fig. 4.5(a) and point B in Fig. 4.5(b) show that a decrease in water content is obtained for these stages, even though F is an unloading probe. This is quite typical of the materials which have very

strong memory (high creep potential) and as one day rest was allowed before applying the probe, the sample has a tendency to creep in the same direction before it gets a feel for the probe. The typical variation ABCDE shown in the figure is similar to the variation postulated by Poulos (1964). Another aspect, besides the creeping tendency of the material, which has affected the strain response, is the magnitude of the probe. The total strain response (recovery) in four days given by ABCDE is far less than that of the large probe given by AF. Leonards and Ramaiah (1959) observed the same behaviour in loading consolidation tests. Leonards & Ramaiah's work was done under conditions of zero lateral strain, where the stress path for the stress increment forms a continuation of the main consolidation stress path. On the contrary, the tests described in this investigation are along probes that represent a definite change of direction for the stress path. Fig. 4.5(c) shows the effect of the magnitude of the probe on the strain response observed by Lewin & Burland (1970) for stress path B(θ =249°) test conducted on a material which had very low creep potential. From this, it can therefore, be argued that the effect as observed would vary with the stress path, but the magnitude of stress probe is an important factor in determining the strain pattern in these tests.

4.5 STRESS-STRAIN-VOLUME CHANGE CHARACTERISTICS FOR NORMALLY CONSOLIDATED CLAY

The various stress paths along which tests have been conducted are described in Fig. 3.3 (Chapter 3). Details of tests are given below.

4.5.1 Special Tests

Two tests were conducted which incorporated more than one stress path. These tests were conducted from the basic stress state of $q/p' = 0.4$. In one test (Special test 1), the sample after being anisotropically consolidated along $q/p' = 0.4$, was sheared along stress path D and was then unloaded back upto the starting point (basic stress state). It was then sheared along F_1 stress path (which forms a continuation of the anisotropic line). The magnitude of F_1 stress probe was adjusted as per the definition given in Fig. 3.3. Fig. 4.6 gives complete details of the special test 1, whereas Fig. 4.7 shows the axial and volumetric strains for the stages after the anisotropic consolidation. Fig. 4.6 shows that the stress-strain curve for F_1 stress path is not a continuation of the anisotropic consolidation path, as the measured strains are much less due to the small magnitude of the probe and also due to the different stress history(loading & unloading along D stress path) the sample has experienced before. This confirms partly the conclusions of section 4.4. During unloading along stress path D, volumetric

strains continue in the positive direction. Test results of F_1 stress path are compared with other stress paths later.

Fig. 4.8 shows the details of special test 2. In this test, after the initial anisotropic consolidation of the sample, it is subjected to E_1 stress path (q-constant) and then unloaded back to reach the basic stress state. It is then unloaded along the anisotropic line (F stress path) so that the sample has an OCR of 1.25. From this lightly overconsolidated state, the sample is then subjected to shearing along the conventional drained test (A stress path) and unloaded back to reach the starting point on the anisotropic line. P-constant unloading test was then carried out. This test encompasses a number of stress paths both in the loading & unloading stages. The unloading behaviour is interesting and is separately discussed in this chapter. There is no recovery in the volumetric strains on unloading. Stress paths E_1, F(indirect) and A(OCR = 1.25) are compared with others subsequently.

4.5.2 Test from the Basic Stress State $q/p' = 0.1$.

Fig. 4.9 shows the stress-strain and strain volume-change behaviour for stress path A. This is the only test conducted from this state. As this test was conducted in the later stages when the originally prepared slurry was over, this sample is likely to differ slightly from the samples of the other series. Dotted lines in the figure, give

the results of the conventional drained test by Varadarajan, conducted on isotropically consolidated samples of this clay. The two curves, show approximately similar behaviour. This is expected as the basic stress state $q/p' = 0.1$ is close to the anisotropic line.

4.5.3. Results of Tests conducted along various Stress Paths from the Basic Stress States $q/p' = 0.2$ & 0.4.

These tests form the main set of tests conducted in this investigation. Figs. 4.10 and 4.11 give the comparative stress-strain volume change response of various stress paths conducted from the two basic stress states. Inset diagrams in the two figures give details of the stress paths whose strains are comparatively smaller and cannot be accurately read & compared along with others in these figures. The stress-strain curves are similar for the two basic stress states. F_1 (indirect) test is discussed in Figs. 4.6 & 4.7 whereas E_1 and F (indirect) tests are discussed in Fig. 4.8. Positive volume changes show compression of the sample. Stress path F tests were compared in other form in Fig. 4.5. The different responses observed for F tests in Fig. 4.11 are due to the different histories the samples had acquired.

These tests take about 10 days upto the final stage of shearing for different stress paths. Assuming the maximum possible leakage rate [Poulos (1964)] of 0.00046% per day, the expected leakage for 10 days period would then be 0.0046%. This leakage rate, if,

assumed for these tests, would accordingly affect the volumetric strain response. The measured volumetric strains for loading stress paths, such as A, B, G_1, E_1, F_1 will reduce after correction for leakage, whereas for unloading stress paths, such as C,D,E,F,G, these will increase. Again, due to the "creeping tendency" of this clay and due to the small magnitude of the probe in the initial stages, a delayed stress-strain response is observed atleast for the first two stages along each probe.

B and E_1 stress path tests have special significance, as the former is a pure shear test and the latter a consolidation test from an anisotropic line. It is evident from the strain response obtained for these two tests, that volumetric and shear strains are observed in each of the two tests, thus confirming the dilatant nature of the soil behaviour. This observed behaviour is incompatible with the elastic theory concepts where there is complete uncoupling of the hydrostatic (spherical) components of the two tensors from the deviatoric components. Strain response for other stress paths (between B and E_1) are in between the responses observed from B and E_1 tests individually. It would thus appear interesting to predict the strain response for the stress-paths like A, G_1 (between B & E_1) by combining in some form the responses obtained from E_1 & B stress paths individually. The similar prediction, in other sectors appears feasible. This will be discussed in detail in Chapter 6.

A comparative study of the volumetric, shear, axial and lateral strains obtained from various tests (for the four stages) has been shown in Figs. 4.12 & 4.13. The comparison of axial and volumetric strains (for the final increment) for the two basic stress states is shown in Fig. 4.14. For volumetric strains, there is a symmetry about $\theta = 270°$, which corresponds to the conventional drained test (the direction of maximum change in σ_1'); whereas, for axial, shear and lateral strains, the symmetry is about $\theta = 246°$, which corresponds to the stress path B test (pure shear test). This symmetry is more marked for tests from the basic stress state of $q/p' = 0.4$ given in Fig. 4.13. There is not much significance in the direction of the stress probe which gives maximum strain response, as this could be due to the increased stress ratio reached depending upon the stress path, besides the magnitude of the probe. Fig. 4.13 shows, in addition, constant η lines (shown dotted) which actually describe the observed strain pattern on the lines just discussed. Fig. 4.14 indicates that the axial strain response is higher, in general for the tests from the basic stress state of $q/p' = 0.4$. The volumetric strains are not very much different for the two basic stress states. The observed pattern of strains is similar to Lewin & Burland (1970).

4.5.4 Observed Lateral Strains and estimation of K_o

It is very difficult to get the stress probe directions which give zero lateral strains from the figures. K_o for this clay can,

however, be indirectly assessed from the results of anisotropic consolidation tests by observing the axial and volumetric strains in these tests. Fig. 4.3 (c) shows these results. Results of tests along basic stress states of 0.1 & 0.2 gave $\frac{\Delta V/V}{\Delta L/L}$ ratio more than 1, whereas, for the basic stress state of 0.4, this ratio was less than 1. Hence, K_o has been obtained by interpolation as shown. Estimated K_o for this clay is 0.685. K_o has been discussed in detail in Chapter 6.

4.6 STRESS-STRAIN-VOLUME CHANGE CHARACTERISTICS FOR LIGHTLY OVER CONSOLIDATED CLAY

Test results of stress paths A, B & C from a lightly over-consolidated state (OCR = 1.25) are described along with a special conventional drained test conducted on a sample, which was allowed 20 day "prior" creep.

Fig. 4.15 (b) compares the stress-strain-volume change behaviour for the three stress paths. The order of the curves is similar to the one obtained for corresponding tests on normally consolidated clay. Stress path C gives negative volumetric strains (expansion) as compared to other two, which show contraction in volume. The curves show that initial tangent modulus for C stress path is much higher than that for B stress path. Similarly, it is much higher for B than A. The test for B stress path indicates that during the last increment, as the stress ratio was closer to the failure value,

undrained failure took place. This corresponds to $\eta = 0.93$. The last increment should have been applied slowly to avoid this.

Figs. 4.15(a), 4.16(a) & 4.16(b) show a comparative study of the stress-strain behaviour of normally & lightly overconsolidated samples for different stress paths. Lightly overconsolidated stress-strain behaviour is altogether different. Figures clearly indicate that the drained modulus of a lightly overconsolidated clay for a given stress path is much more than the corresponding value for the normally consolidated clay. In order to correlate the behaviour of lightly overconsolidated samples as described above with the relevant insitu behaviour, the special conventional drained test results (for 20 day aged sample) are also shown in Fig. 4.16(a). It is apparent from this figure, that the two stress-strain curves (A stress path from OCR = 1.25 and from 20 day aged sample) are close to each other, thus confirming the viewpoint that a good idea of the insitu behaviour (atleast qualitatively) of a soft marine clay can be had from tests on lightly overconsolidated samples. The reason why the special drained test shows a stiffened stress-strain response is obvious from the work of Bjerrum (1967). Fig. 4.16 (a) also shows an 'indirect' A stress path test (see special test 2). The stress-strain curve is different due to the different stress history experienced.

This study on lightly overconsolidated samples gives substantial weightage to the concepts provided by Parry & Nadarajah (1974), who

have described undrained tests from various OCR values (in the light overconsolidation range).

4.7 BEHAVIOUR DURING UNLOADING ALONG VARIOUS STRESS PATHS

After probing along different directions, the samples were unloaded to get an idea about recoverable strains (volumetric & shear) associated with various stress paths. A knowledge of the actual recoverable strains helps in assessing the predictions based on the theory of plasticity (for example, work of Roscoe & Co-Workers). Figs. 4.17 through 4.20 compare loading-unloading curves of stress paths A, B, C, G_1 respectively for the two basic stress states, whereas Fig. 4.21 gives curves for stress path E_1 (for the basic stress state of $q/p' = 0.4$). Loading & unloading curves for A and B stress paths for the lightly overconsolidated samples are given in Fig. 4.22. Shear stresses have been plotted against shear strains to know the recovery in shear strains during unloading. The unloading portions of the stress-strain-volume change curves for various stress paths have been compared in Figs. 4.23 and 4.24 for the two basic stress states.

It can be observed from the figures discussed that there is no recovery in the volumetric strains. On the contrary volumetric strains go on in the same direction. There is small recovery in the shear strains for the last stages of the probes. It can, therefore, be argued that for small probes (the first two stages of

each stress path), the assumption of irrecoverable shear and volumetric strain is valid. The recovery of strains is apparently a function of the stress level from where unloading is carried out. It is, however, obvious that the above assumptions are not valid if unloading is done to very low stress levels. These also substantiate the findings of Lewin & Burland's (1970) study on small probes.

4.8 EVALUATION OF VARIOUS DRAINED MODULI FROM THE TRIAXIAL TEST RESULTS USING THEORY OF ELASTICITY

With the development of finite element technique, it has become feasible now to analyse problems associated with footings, retaining structures, excavations etc., taking into consideration the nonlinear stress path dependent stress-strain behaviour of each soil element. It is shown in this chapter that the stress-strain relationships are highly stress path dependent. The various moduli required for an analysis using the finite element technique can be evaluated using the theory of elasticity.

The theory of elasticity equations for an isotropic homogeneous material relating elastic constants with stresses and strains can be written as :

$$E' = \frac{(\Delta\sigma_1' + 2\Delta\sigma_3')\,(\Delta\sigma_1' - \Delta\sigma_3')}{\Delta\sigma_3'(\Delta\varepsilon_1 - 2\Delta\varepsilon_3) + \Delta\sigma_1'\,\Delta\varepsilon_1} \tag{4.1}$$

$$\nu' = \frac{\Delta\sigma_3'\,\Delta\varepsilon_1 - \Delta\sigma_1'\,\Delta\varepsilon_3}{\Delta\sigma_3'\,(\Delta\varepsilon_1 - 2\Delta\varepsilon_3) + \Delta\sigma_1'\,\Delta\varepsilon_1} \tag{4.2}$$

$$G' = \frac{\Delta\sigma_1' - \Delta\sigma_3'}{2(\Delta\varepsilon_1 - \Delta\varepsilon_3)} \tag{4.3}$$

$$K' = \frac{\Delta\sigma_1' + 2\Delta\sigma_3'}{3(\Delta\varepsilon_1 + 2\Delta\varepsilon_3)} \qquad (4.4)$$

These relations are derived on the assumption that elastic constants do not change during the infinitesimal stress and strain changes.

Where, E' is the effective Young's modulus

ν' is the effective Poisson's ratio

G' is the effective shear modulus

K' is the effective bulk modulus

$\Delta\sigma_1'$ and $\Delta\sigma_3'$ are the changes in major and minor effective principal stresses respectively

$\Delta\varepsilon_1$ and $\Delta\varepsilon_3$ are the changes in major and minor principal strains respectively

$\Delta\varepsilon_3$ is given by $\Delta\varepsilon_3 = \frac{1}{2}\left(\frac{\Delta V}{V} - \Delta\varepsilon_1\right)$ (4.5)

where $\frac{\Delta V}{V}$ is the volumetric strain during shear.

Using equations (4.1) through (4.4), the values of E', ν', G' and K' for any stress path can be evaluated. Table 4.1 gives these relationships.

Tables 4.2 and 4.3 list the values of the elastic parameters computed from experimental values for various stress paths using Table 4.1. These are the initial tangent moduli values corresponding to the second stage of each stress path (P' = 23.3 psi = constant at the start of shearing for all tests). All these results correspond to the basic stress state of $q/p' = 0.4$ (this, being close to k_o of

the clay, is more relevant). The elastic parameters for isotropically consolidated samples of the clay are listed in Table 4.4.

The variation of the elastic parameters, E', G', ν' and K' with stress path, both for normally and lightly overconsolidated samples, has been shown in Figs. 4.25 and 4.26. It is clear from these figures and the tables that the drained moduli are highly dependent on the stress path. There is a difference of more than a log cycle in the moduli values for different stress paths. Lightly overconsolidated samples show very high values of the parameters (E', G', K') as compared to the corresponding values for normally consolidated samples e.g., for A stress path,

$$\frac{E'(\text{O.C.C.})}{E'(\text{N.C.C.})} = 3.3 \text{ and } \frac{\nu'(\text{O.C.C.})}{\nu'(\text{N.C.C.})} = 0.36 .$$

Comparison of Tables 4.2 and 4.3 with Table 4.4 shows that the difference in the elastic parameters for this clay with the field consolidation (k_o) history is marked as compared to the behaviour observed for samples with isotropic consolidation history.

It can, therefore, be argued that relevant moduli values (with k_o-consolidation history and light overconsolidation) must be used in the analysis of any stress-deformation problem; otherwise the predictions are going to be erroneous. This confirms the findings of Bjerrum (1973), Parry & Nadarajah (1974), Ladd (1964,1966), Simons and Som (1969), Simons (1972), Vaid & Campanella (1974), Berre and Bjerrum (1973).

4.8.1 Anomalous Behaviour of the Moduli Parameters for a Few Stress Paths using Theory of Elasticity

It is evident from the results presented in Tables 4.2 through 4.4 that inadmissible values of the elastic constants E', ν' and K' are obtained for stress path B; and ν', G', E' for stress paths E & E_1. To appreciate this anomaly for these stress paths, it is necessary to recall that the formulations given in equations 4.1 through 4.4, are for ideal elastic solids for which volume change due to changes in deviatoric or shear stresses (q) is zero and all volume changes are due to changes in volumetric or mean stress (P') component only. On the contrary, the results presented in Figs. 4.7 & 4.8 indicate that the total volume change is due to changes in both shear & hydrostatic stresses. For B stress path (which is a pure shear test) or those in its vicinity, the changes in P' are either zero or very small, and most of the volume change is primarily due to changes in shear stress (q). Similarly, for stress paths E and E_1 (which are q-constant tests) or those in their vicinity, the changes in q are either zero or very small, and most of the shear strain is primarily due to changes in mean stress P'. Since equations (4.1) through (4.4) do not account for volume changes due to shear stresses or shear strains due to hydrostatic stresses, the use of these equations lead to inadmissible results. This aspect is discussed by Yudhbir et. al. (1975). It is suggested that for B stress path, ν' could be taken

as an average of ν'-values for A and C stress paths. E' and K' can then be determined from the relations :

$$E' = 2G' (1 + \nu') \tag{4.7}$$

and $$E' = 2K' (1 - 2\nu') \tag{4.8}$$

G' can be substituted in Eq. (4.7) from Table 4.2.

For stress path E_1 , assessment of ν' becomes a problem. Theory of elasticity gives a value of 0.5 (valid for undrained case). However, seeing the variation of ν' with θ in Fig. 4.26, an average value of $\nu' = 0.4$ could be assumed. E' can then be obtained using Eq. (4.8) (K' in this equation can be put from Table 4.2).

It is interesting to compare the ratios $\frac{E'_{ic}}{E'_{iA}}$ and $\frac{\nu'_c}{\nu'_A}$ for three different soils. Table 4.5 gives an idea of the variation in E' & ν' parameters for the two stress paths A & C for different conditions.

Even the use of G' & K' rather than E' & ν', as recommended by Domaschuk & Valliappan (1975) cannot account for the anomalies for the stress paths discussed above.

The above discussions were based on the assumption that soil behaves as an isotropic material. The anisotropy is inherently present in the clays. Even the laboratory samples prepared by resedimentation show the anisotropic behaviour (Lewin, 1971; Balasubramaniam, 1973). However, the observed experimental behaviour in this study, is not influenced by the anisotropy (induced at the

time of sedimentation) as the preshear consolidation pressure was almost eight times the pressure at the time of sedimentation. On the other hand, the inherent anisotropy could be handled by using anisotropic theory of elasticity [as shown by Simons & Som (1969), Henkel (1972) and Wroth (1972)]. The inherent disadvantages, however, remain as the theory of elasticity does not account for the stress path dependent & dilatant behaviour of soils.

4.9 CONCLUSIONS

The results of drained triaxial tests have been presented for various stress paths from the two basic stress states both for normally and lightly overconsolidated clays. The stress path dependent stress-strain-volume change behaviour for anisotropically consolidated clay as presented here, is believed to be the first systematic study of the behavioural aspects done on a natural marine clay. Obviously the interpretation of the results of this study has to be made in the light of the "creeping tendency" of this natural clay. The loading stress paths G_1 and E_1 represent the typical paths usually encountered for footings on clays. Stress path C is more relevant for an active earth pressure problem, whereas G stress path is characteristic of an excavation element. B is a typical pure shear stress path.

Anisotropic consolidation tests show that the anisotropic (η) lines are not parallel and the slope λ of the lines is dependent on η. The maximum observed variation in λ was fifteen percent. Again

determination of K for a natural clay is not easy especially when small increments are involved. All this is going to complicate the simple and useful concepts provided by the Cambridge group. However, a detailed study concerning the evaluation of λ and K is warranted.

Twenty day rest period for the clay, during which the delayed consolidation was allowed, resulted in a P_c/P_o ratio of 1.25 to 1.3. Conventional drained test on a sample, which had probably developed this critical pressure, showed a much stiffer strain-strain response. Tests on lightly overconsolidated samples appear atleast, to qualitatively represent the insitu behaviour of a soft marine clay in a much shorter time.

The unloading behaviour along various stress paths studied, showed that the recovery in volumetric and shear strains was very small.

The one day rest period introduced before shearing, resulted in smaller strains than would be expected from a stress probe that was part of a continuous loading process. It is also shown that the magnitude of the probe has a substantial effect on the strain response.

Finally it is shown that the drained modulus is very much a function of the stress path. The effect on drained moduli of the previous stress history (isotropic/anisotropic consolidation; normally/lightly overconsolidated) is marked and it is emphasized that relevant consolidation stress history must be duplicated in the laboratory for any sensible results.

It is pointed out that the use of isotropic theory of elasticity in interpreting the results of some stress paths lead to an anomalous behaviour. This is due to the fact that the theory of elasticity does not handle the stress path dependent, dilatant behaviour of the clay in its formulation. Concepts based on the theory of plasticity would appear to be more powerful (Roscoe and Burland, 1968) and will be discussed in detail in chapter 6.

Table 4.1(a)

Evaluation of the Elastic Parameters for Different Stress Paths.

Elastic Parameters	Stress Path A	Stress Path B	Stress Path C	Stress Path D
	$\Delta\sigma_3' = 0$ $\Delta\sigma_1'$ increasing	$\frac{\Delta\sigma_1' + 2\Delta\sigma_3'}{3} = 0$ or $\Delta\sigma_1' = -2\Delta\sigma_3'$ $\Delta\sigma_3'$ decreasing $\Delta\sigma_1'$ increasing	$\Delta\sigma_1' = 0$ $\Delta\sigma_3'$ decreasing	$\Delta\sigma_1'$ and $\Delta\sigma_3'$ both decreasing such that $\Delta\sigma_1'/\Delta\sigma_3' = 0.4$
E'(in psi)	$\frac{\Delta\sigma_1'}{\Delta\varepsilon_1}$	0*	$\frac{2\Delta\sigma_3'}{(\Delta\varepsilon_1-2\Delta\varepsilon_3)}$	$\frac{1.44\Delta\sigma_3'}{(1.4\Delta\varepsilon_1-2\Delta\varepsilon_3)}$
ν'	$-\frac{\Delta\varepsilon_3}{\Delta\varepsilon_1}$	-1*	$\frac{\Delta\varepsilon_1}{(\Delta\varepsilon_1-2\Delta\varepsilon_3)}$	$\frac{(\Delta\varepsilon_1-0.4\Delta\varepsilon_3)}{(1.4\Delta\varepsilon_1-2\Delta\varepsilon_3)}$
G'(in psi)	$\frac{\Delta\sigma_1'}{2(\Delta\varepsilon_1-\Delta\varepsilon_3)}$	$\frac{3}{2}\frac{\Delta\sigma_3'}{(\Delta\varepsilon_1-\Delta\varepsilon_3)}$	$\frac{\Delta\sigma_3'}{2(\Delta\varepsilon_1-\Delta\varepsilon_3)}$	$\frac{0.3\Delta\sigma_3'}{(\Delta\varepsilon_1-\Delta\varepsilon_3)}$
K'(in psi)	$\frac{\Delta\sigma_1'}{3(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	0*	$-\frac{2\Delta\sigma_3'}{3(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	$\frac{-0.8\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$

(*Inadmissible values)

Table 4.1(b)

Evaluation of the Elastic Parameters for Different Stress Paths.

Elastic Parameters	Stress Path				
	E	F	G	E_1	G_1
	$\Delta\sigma_1' = \Delta\sigma_3'$ both decreasing	$\Delta\sigma_1' + \Delta\sigma_3'$ both decrease such that $\Delta\sigma_1' = 1.46\Delta\sigma_3'$ $(q/p' = 0.4)$	$\Delta\sigma_1' = 2\Delta\sigma_3'$ both decreasing	$\Delta\sigma_1' = \Delta\sigma_3'$ both increasing	$\Delta\sigma_1' = 2\Delta\sigma_3'$ both increasing
E' (in psi)	0*	$\frac{0.795\Delta\sigma_3'}{(1.23\Delta\varepsilon_1-\Delta\varepsilon_3)}$	$\frac{-4\Delta\sigma_3'}{(3\Delta\varepsilon_1-2\Delta\varepsilon_3)}$	0*	$\frac{4\Delta\sigma_3'}{(3\Delta\varepsilon_1-2\Delta\varepsilon_3)}$
ν'	0.5	$\frac{(\Delta\varepsilon_1-1.46\Delta\varepsilon_3)}{(2.46\Delta\varepsilon_1-2\Delta\varepsilon_3)}$	$\frac{(\Delta\varepsilon_1-2\Delta\varepsilon_3)}{(3\Delta\varepsilon_1-2\Delta\varepsilon_3)}$	0.5	$\frac{(\Delta\varepsilon_1-2\Delta\varepsilon_3)}{(3\Delta\varepsilon_1-2\Delta\varepsilon_3)}$
G' (in psi)	0*	$\frac{-0.46\Delta\sigma_3'}{2(\Delta\varepsilon_1-\Delta\varepsilon_3)}$	$\frac{-\Delta\sigma_3'}{2(\Delta\varepsilon_1-\Delta\varepsilon_3)}$	0*	$\frac{\Delta\sigma_3'}{2(\Delta\varepsilon_1-\Delta\varepsilon_3)}$
K' (in psi)	$\frac{-\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	$\frac{-1.15\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	$-\frac{4}{3}\frac{\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	$\frac{\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$	$\frac{4}{3}\cdot\frac{\Delta\sigma_3'}{(\Delta\varepsilon_1+2\Delta\varepsilon_3)}$

(*Inadmissible values)

Table 4.2

Elastic Parameters for Rann of Kutch Clay with Anisotropic Stress History.

Basic Stress State ($q/p' = 0.4$)

Elastic Parameters	Stress Path					
	E_1	G_1	A	B	C	D
E'(in psi)	0	295	446	0*	2,290	26,800
ν'	0.5	0.306	0.234	-1*	0.495	0.55
G'(in psi)	0	113	228	168	565	11,200
K'(in psi)	425	252	144	0*	25,400	3,020

(*Inadmissible values)

Table 4.3

Elastic Parameters for Lightly Overconsolidated Rann of Kutch Clay (OCR = 1.25).

Basic Stress State ($q/p' = 0.4$)

Elastic Parameters	Stress Path		
	A	B	C
E'(in psi)	1,486	0*	11,300
ν'	0.085	-1*	0.274
G'(in psi)	685	1,000	4,040
K'(in psi)	596	0*	7,580

(* Inadmissible values)

Table 4.4

Elastic Parameters for Rann of Kutch Clay with Isotropic Stress History.

[Taken from Yudhbir et. al. (1975)]

(Preshear Confining Pressure = 15.6 psi)

Elastic parameters	Stress Path			
	A	B	C	D
E'(in psi)	240	0*	985	1,540
ν'	0.15	-1*	0.46	0.41
G'(in psi)	130	160	410	1,600
K'(in psi)	103	0*	2,320	555

(*' Inadmissible values)

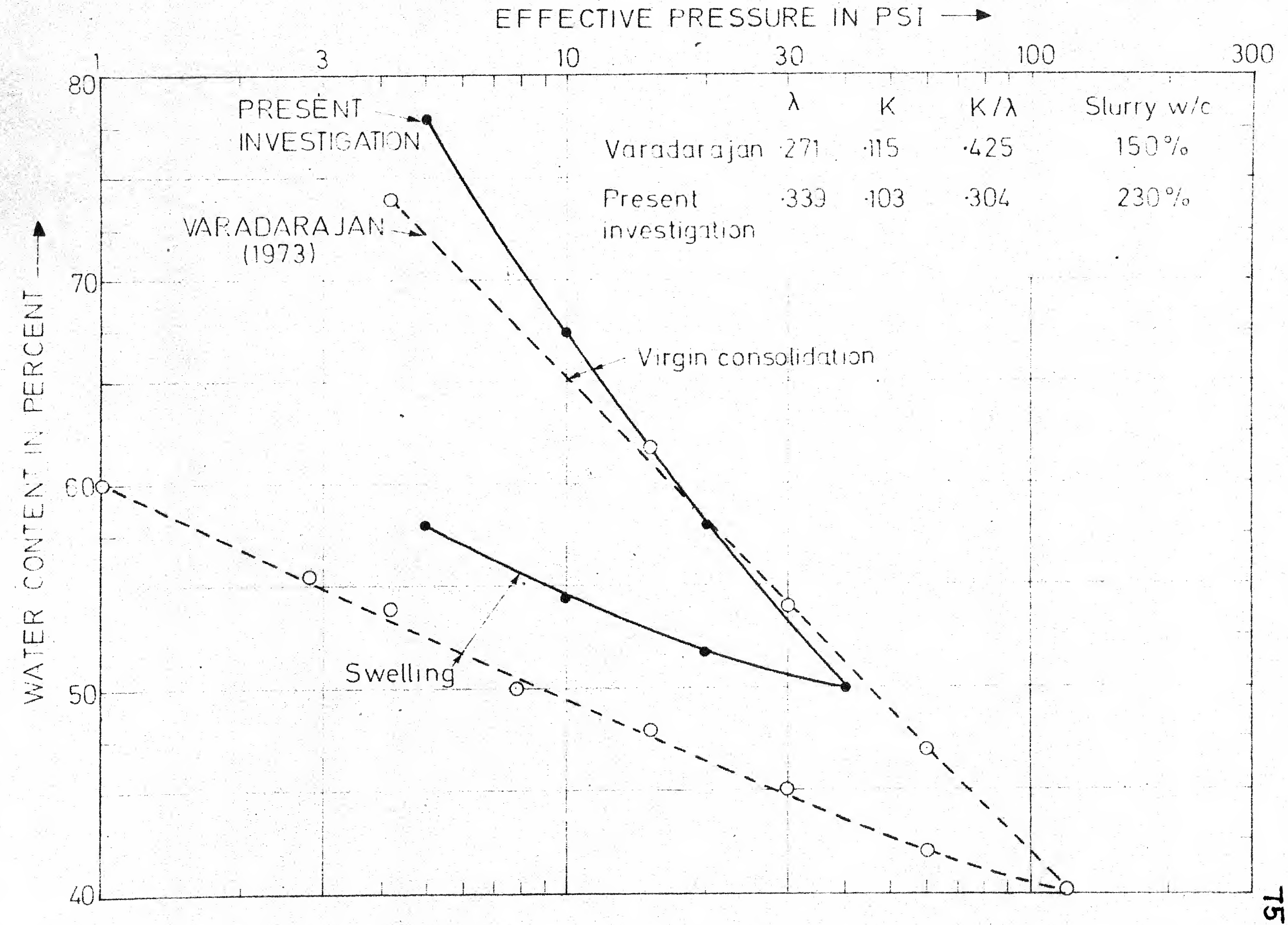
Table 4.5

Comparison of Young's Modulus & Poisson's Ratio for Different Conditions.

Soil type	$\frac{E'_{ic}}{E'_{iA}}$ *	$\frac{\nu'_c}{\nu'_A}$ **
1. Rann of Kutch clay (P.I.=49%)		
N.C.C.(isotropic stress history)	4	3
N.C.C.(anisotropic stress history)	5.13	2.12
O.C.R.=1.25(anisotropic stress history)	7.6	3.22
2. N.C.C. Weald clay (P.I.=25%)	1.6	2.1
3. Loose Sand (relative density = 33%) [Breth et. al. (1973)]	1.3	10

* E'_{iA} & E'_{ic} are the initial tangent Young's moduli for stress paths A and C respectively.

**ν'_A & ν'_C are the Poisson's ratio for stress paths A & C respectively.



FIG·4·1 ISOTROPIC CONSOLIDATION OF RANN OF KUTCH CLAY

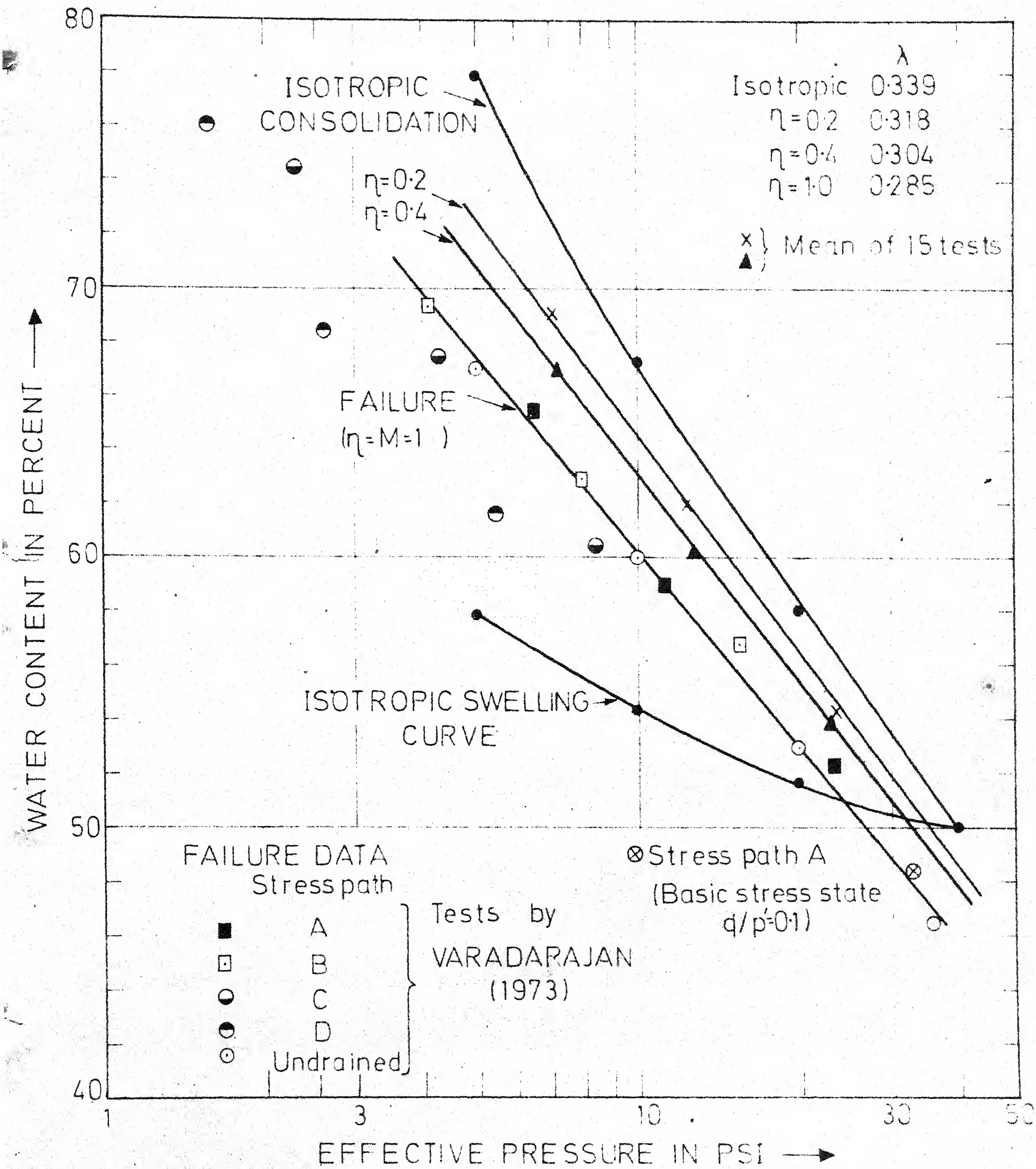


FIG.4.2 ISOTROPIC & ANISOTROPIC CONSOLIDATION OF RANN OF KUTCH CLAY

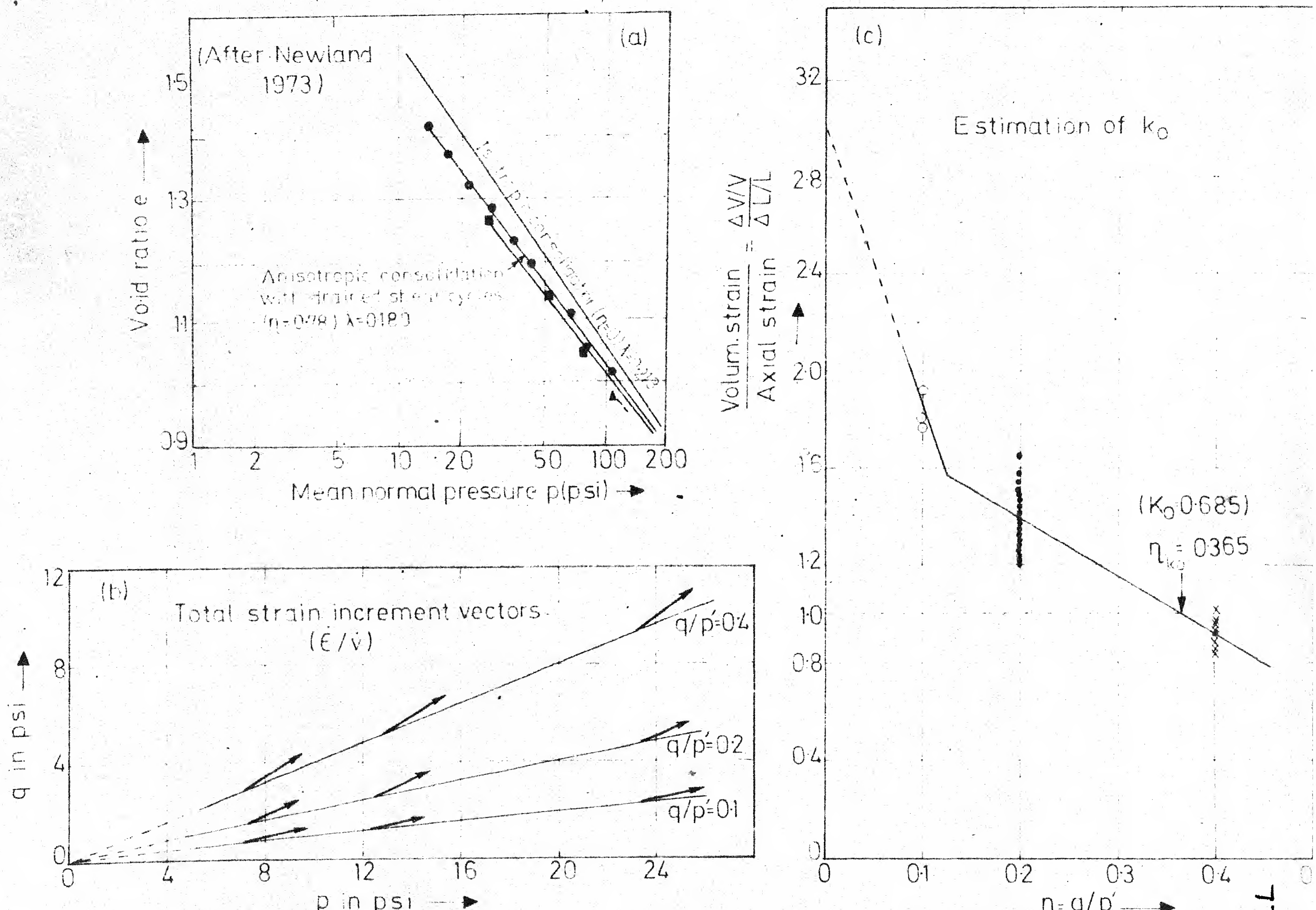


FIG.4.3 ANISOTROPIC CONSOLIDATION TEST RESULTS

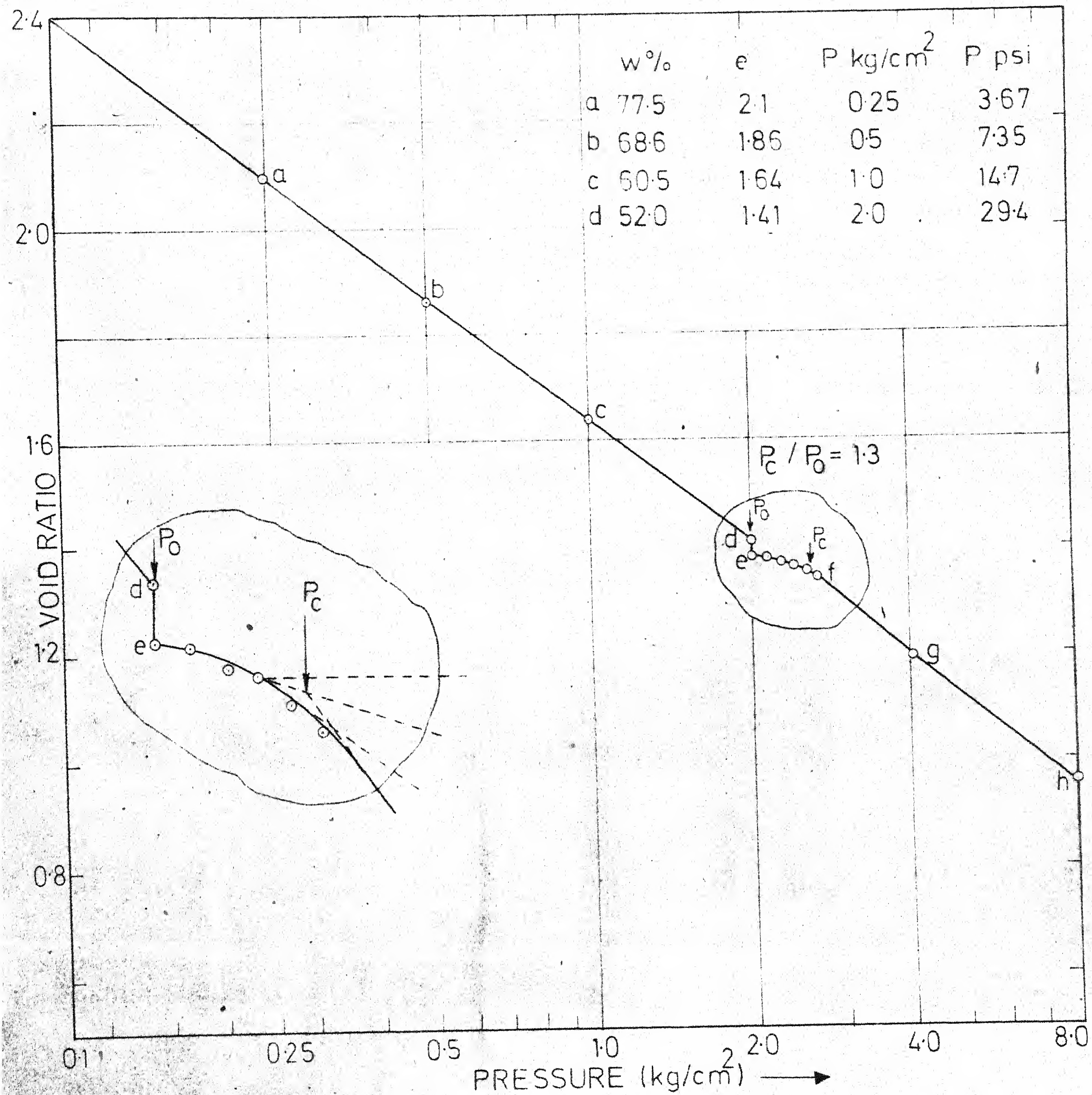
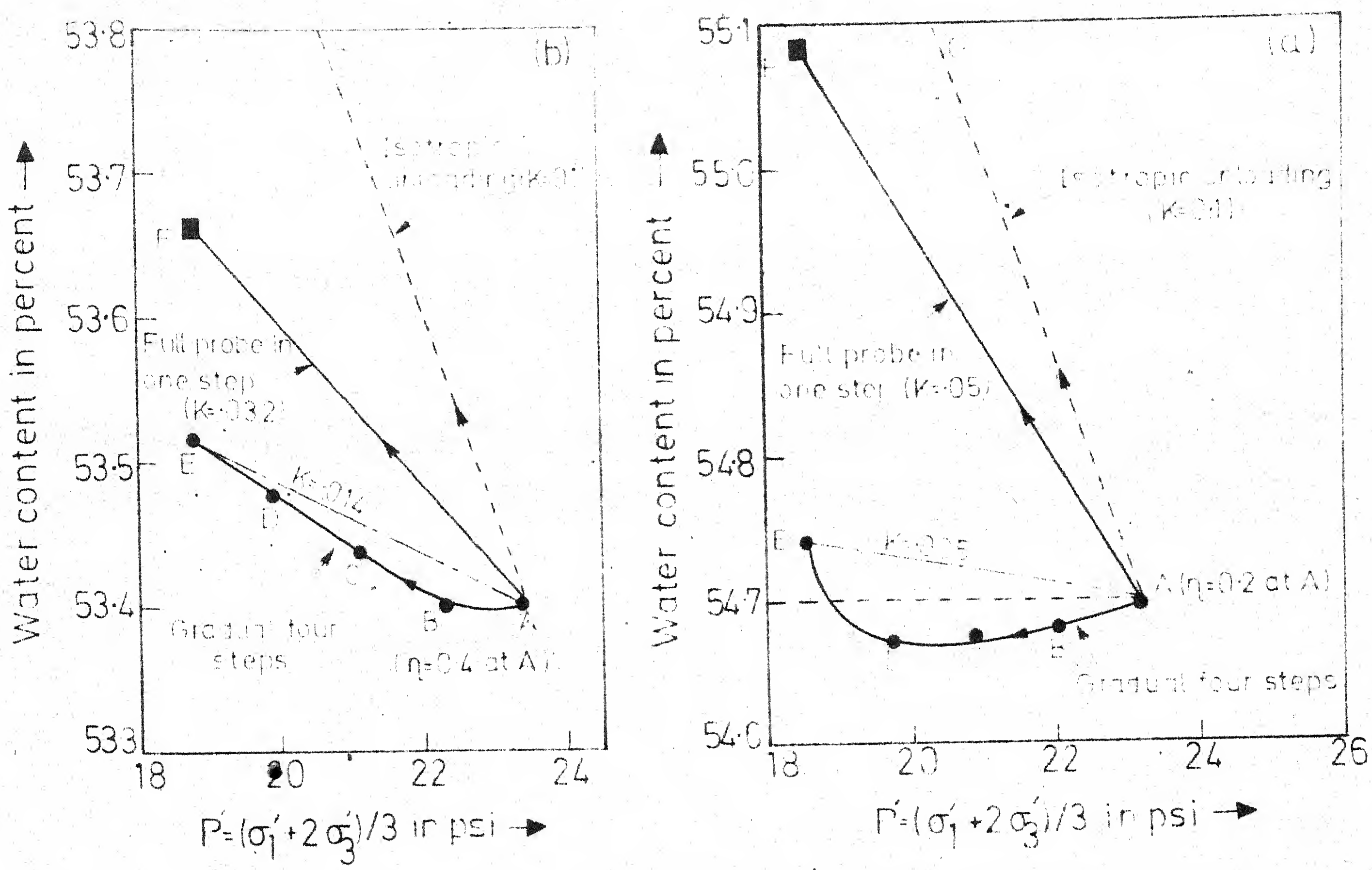
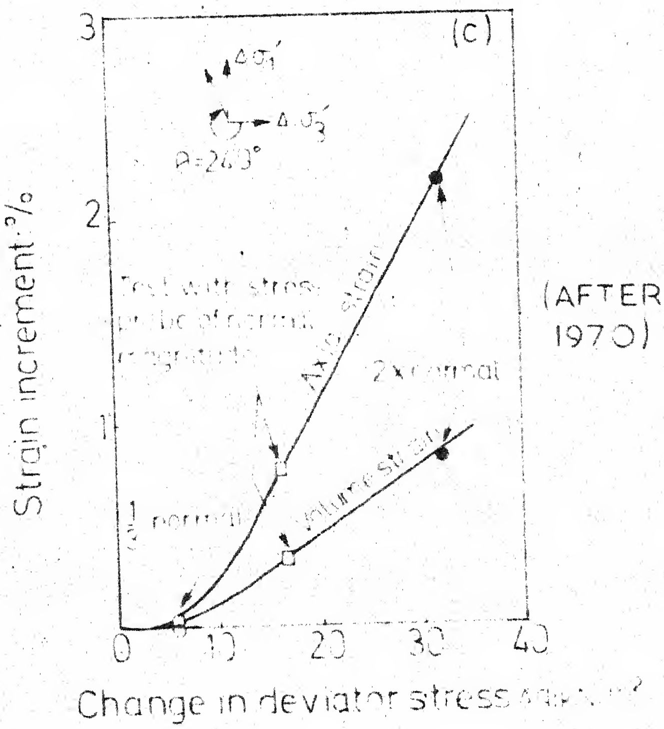


FIG.4·4 CONSOLIDATION TEST - P_c DETERMINATION

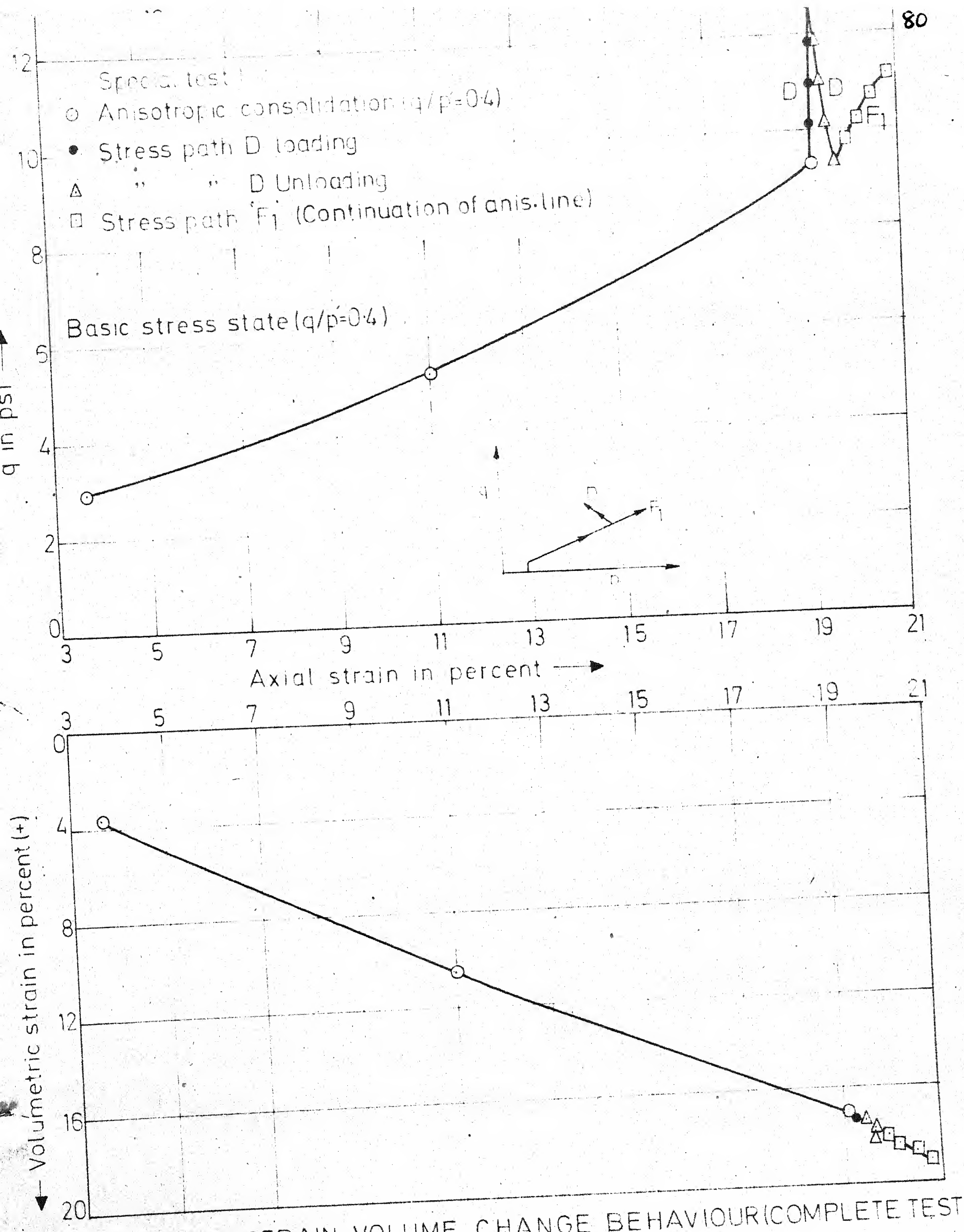


UNLOADING ALONG ANISOTROPIC LINE SHOWING THE EFFECT OF MAGNITUDE OF PROBE (STRESS PATH 'F')



(AFTER LEWIN & BURLAND 1970)

FIG. 4.5 EFFECT OF VARYING MAGNITUDE OF STRESS INCREMENT



FIG·4·6 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR (COMPLETE TEST DETAILS)

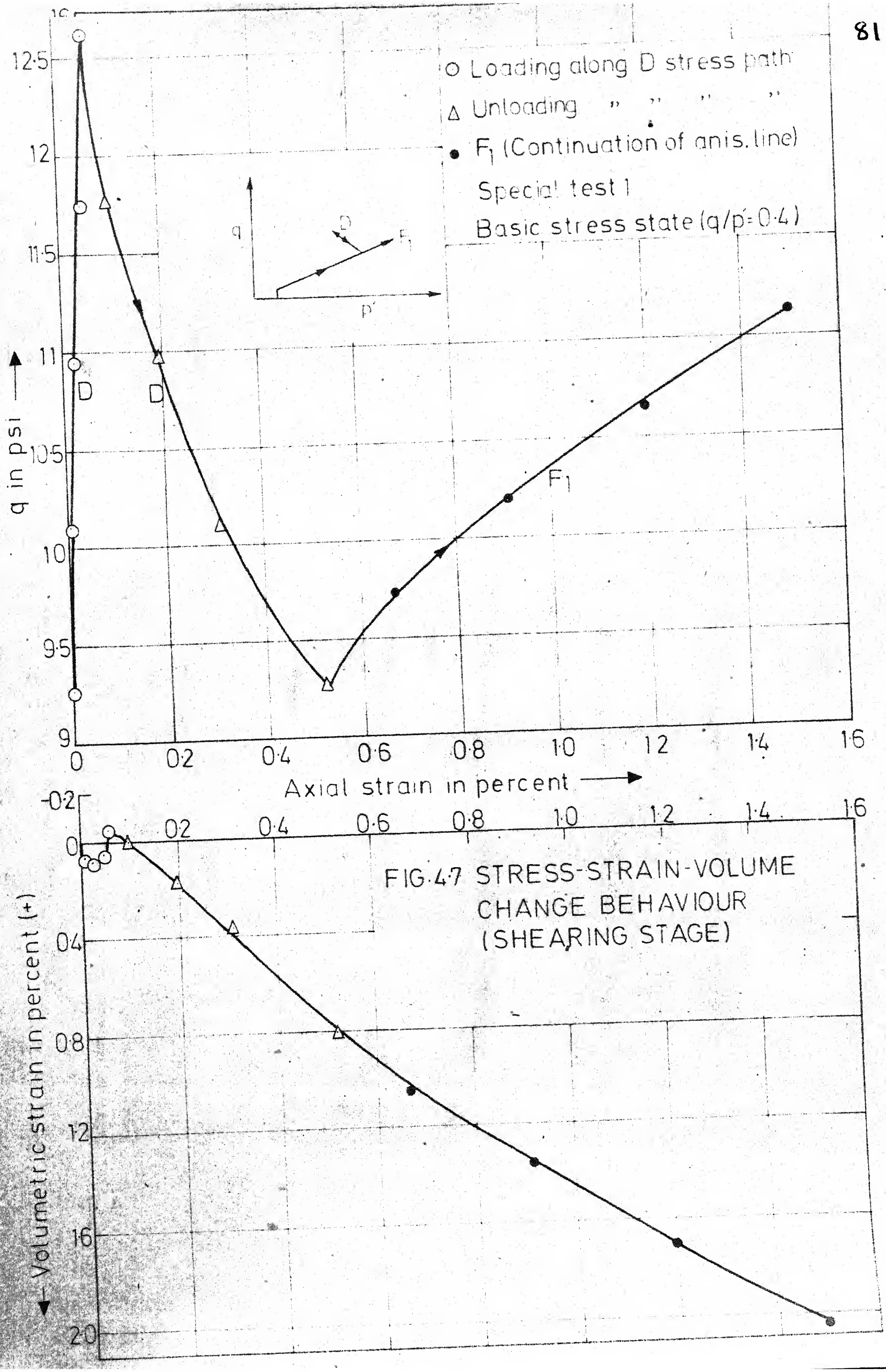


FIG. 4.7 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR (SHEARING STAGE)

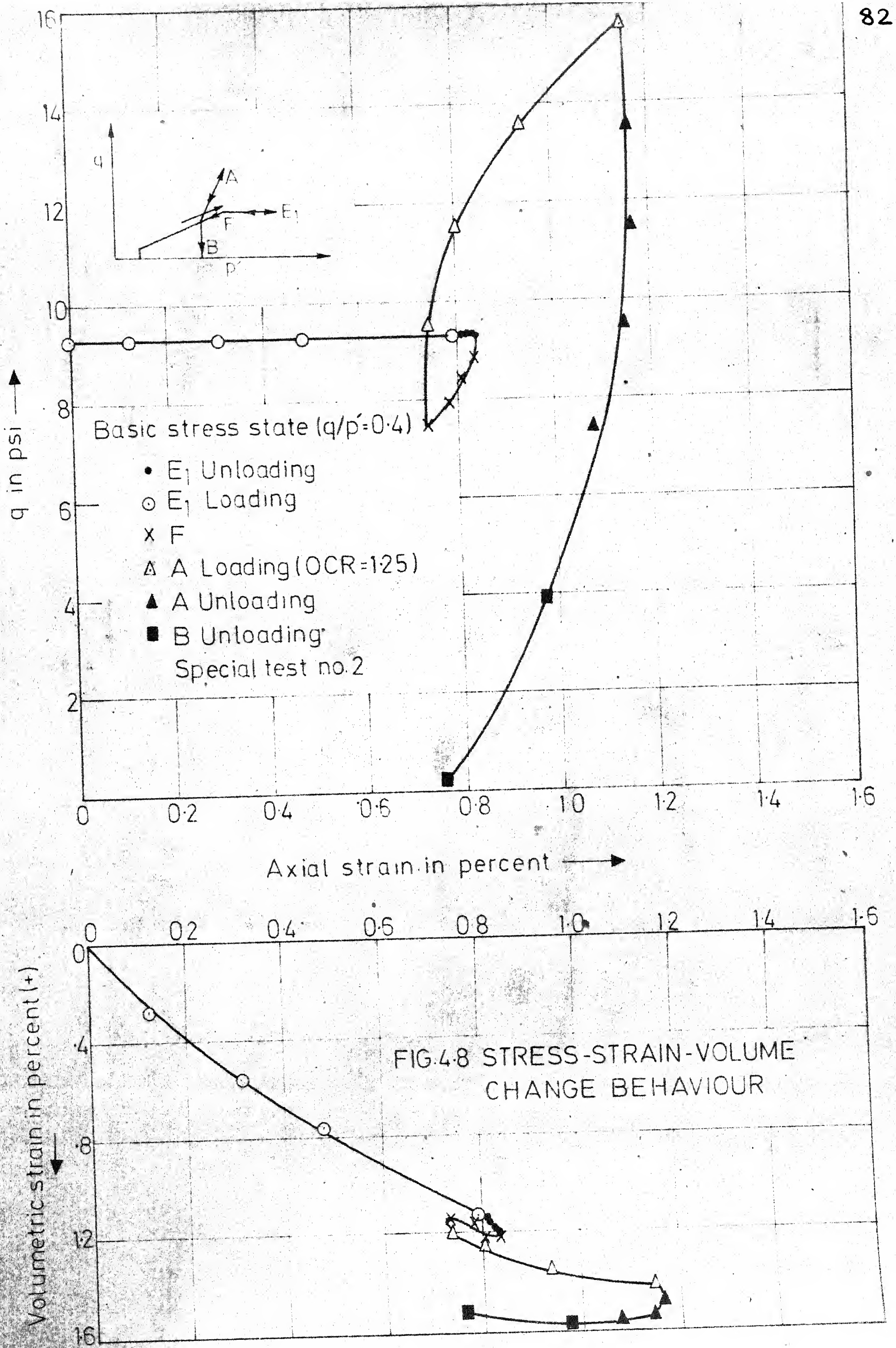


FIG.4·8 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR

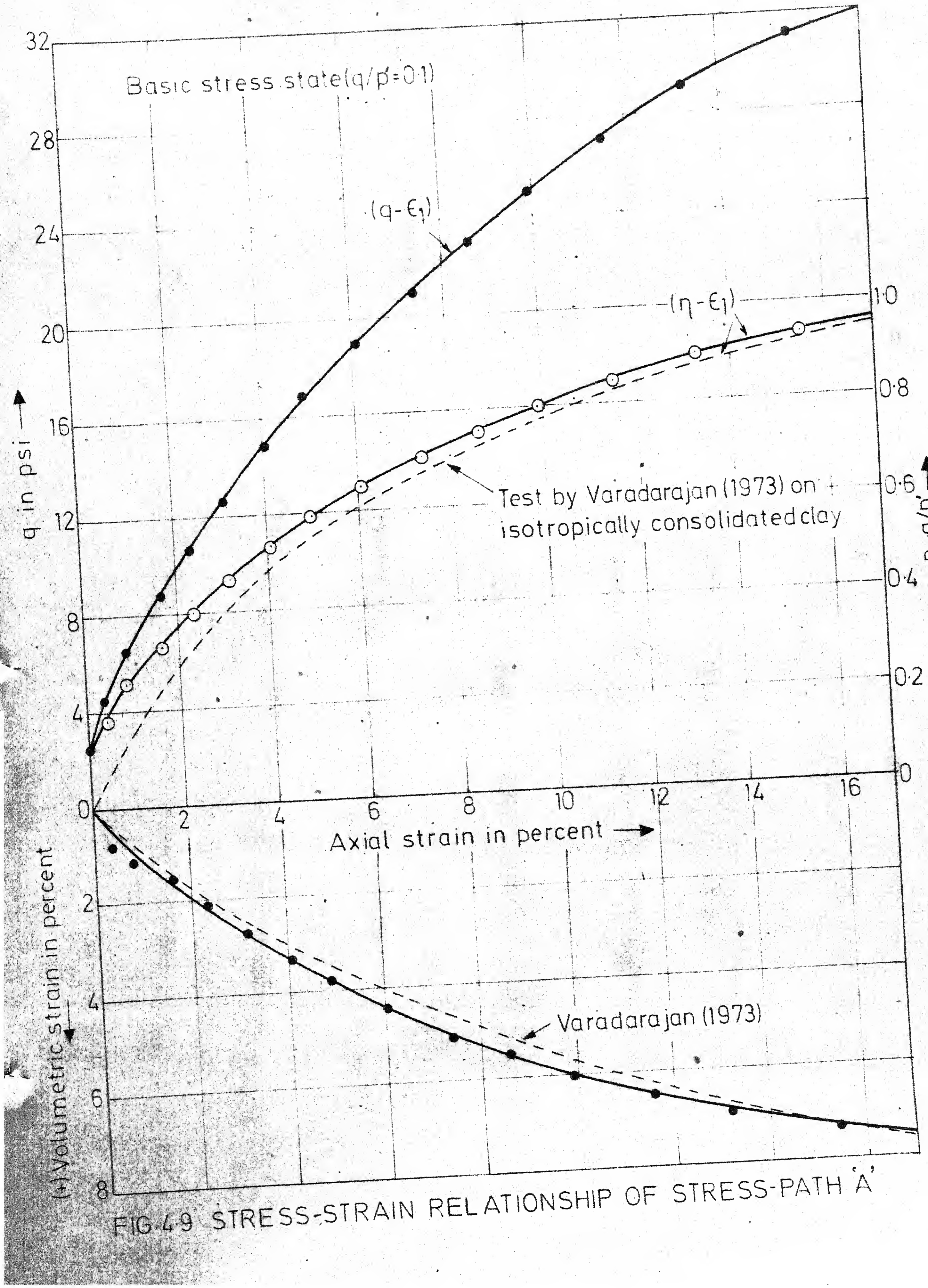


FIG.4.9 STRESS-STRAIN RELATIONSHIP OF STRESS-PATH 'A'

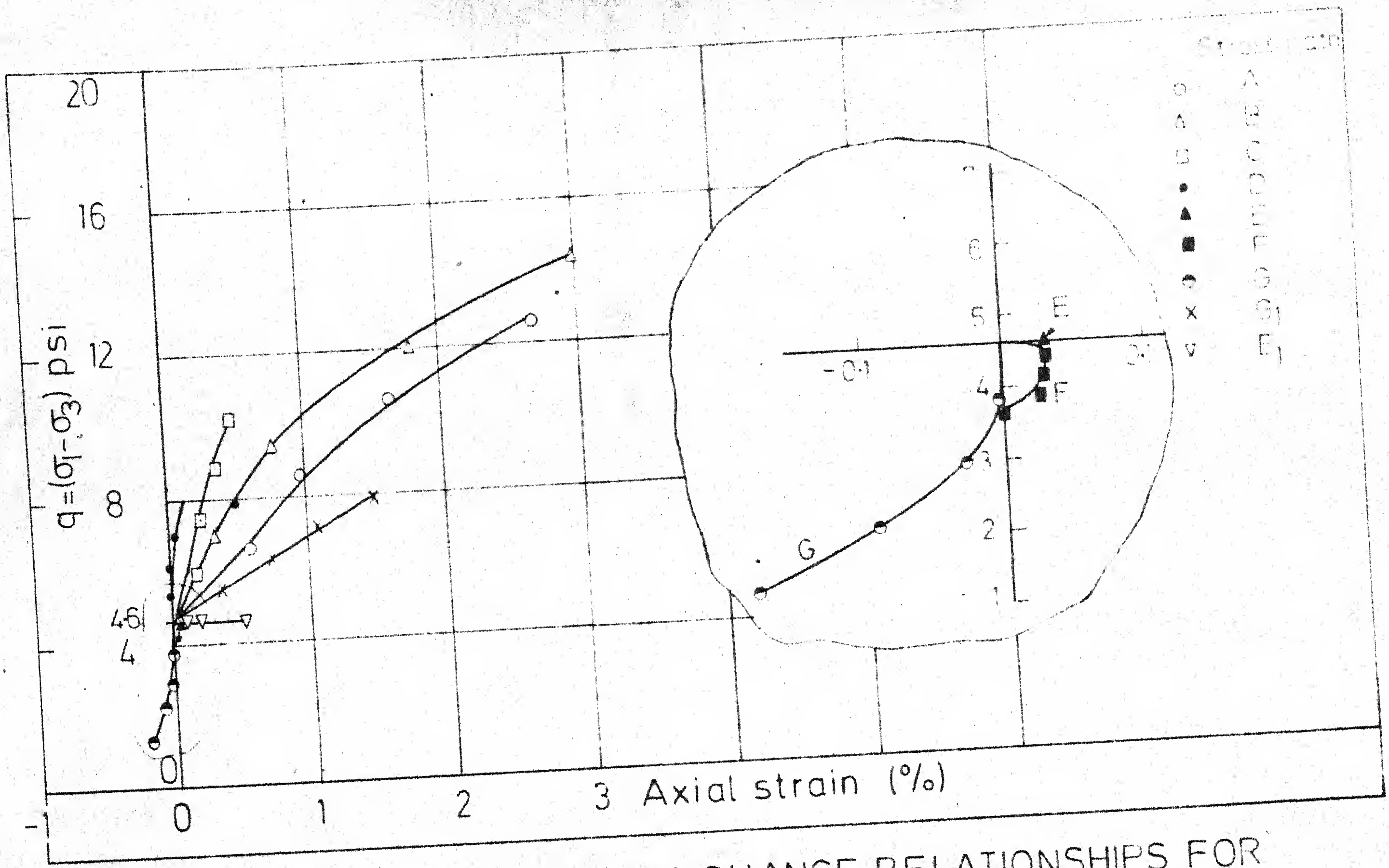
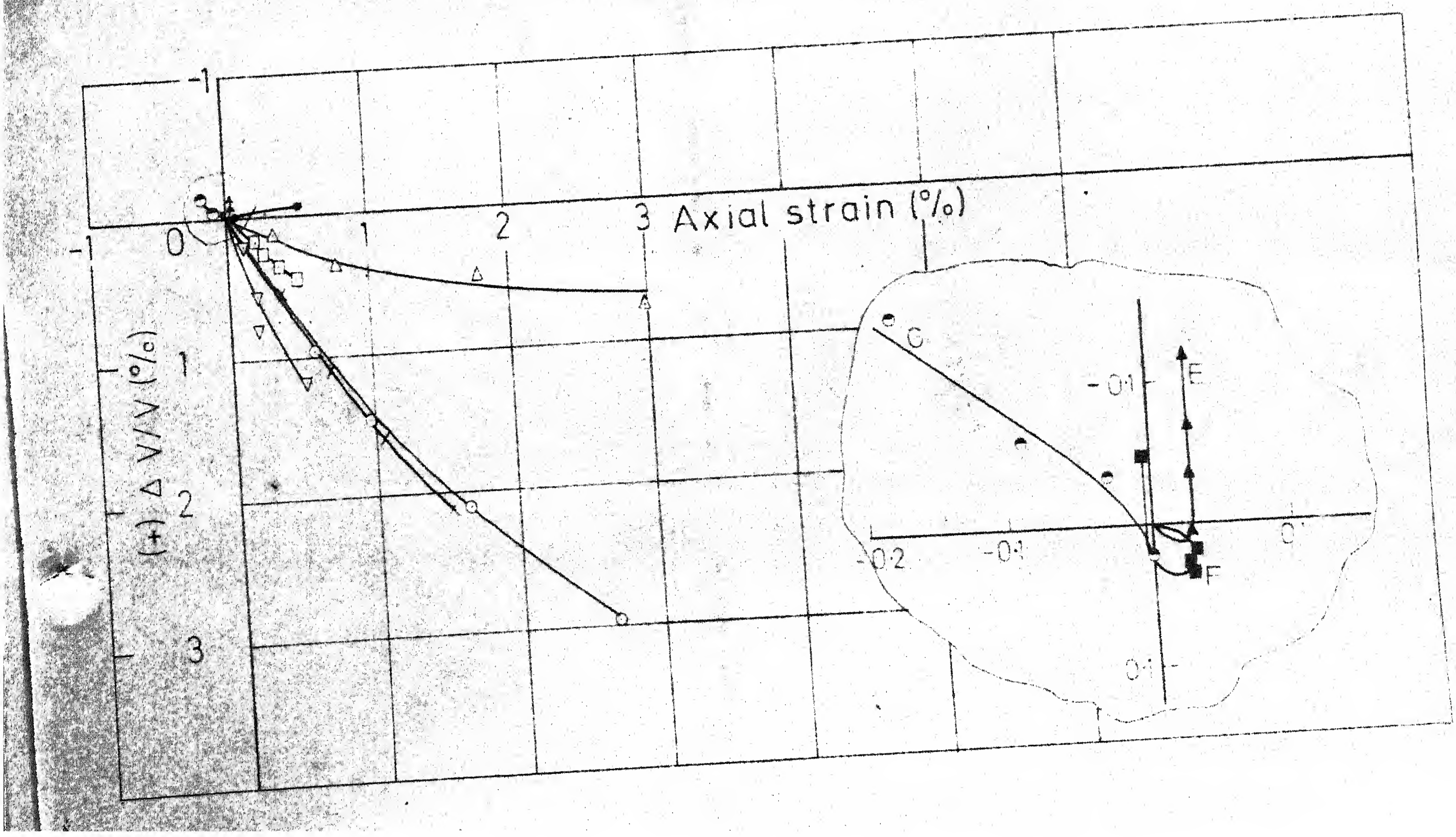
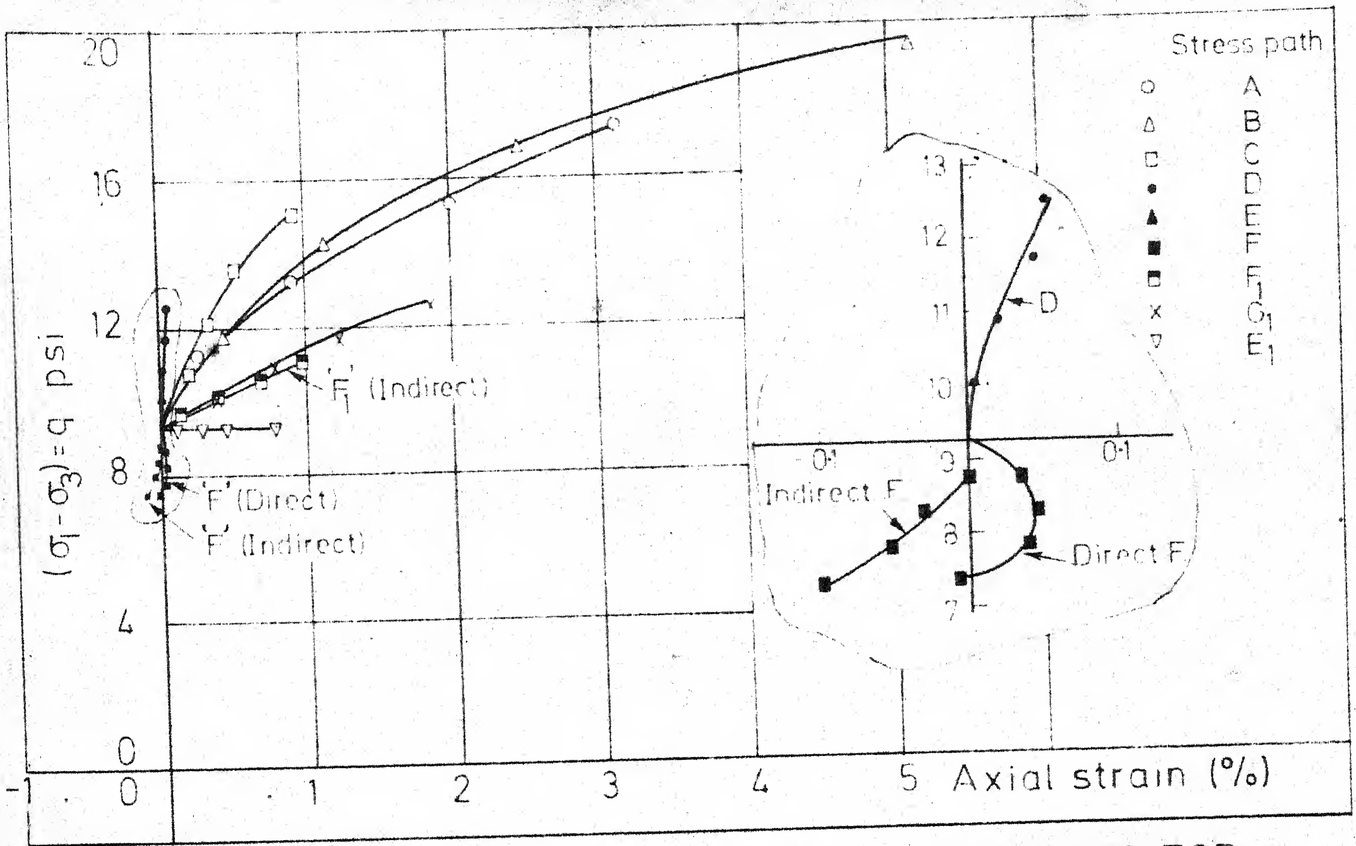


FIG. 4.10 STRESS-STRAIN-VOLUME CHANGE RELATIONSHIPS FOR VARIOUS STRESS PATHS

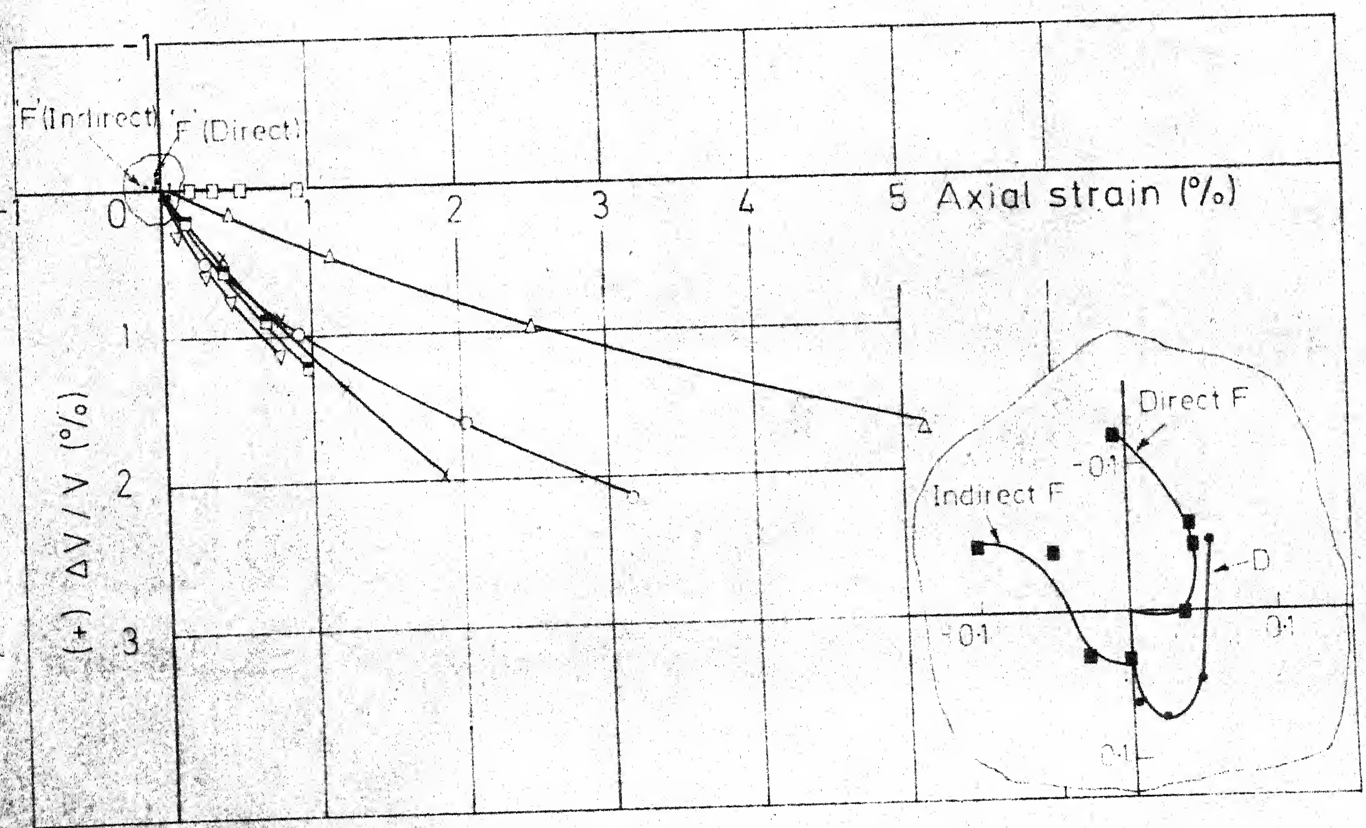
Basic stress state $(q/p' = 0.2)$





FIG·4·11 STRESS-STRAIN-VOLUME CHANGE RELATIONSHIPS FOR VARIOUS STRESS PATHS

Basic stress state ($q/p'=0{\cdot}4$)



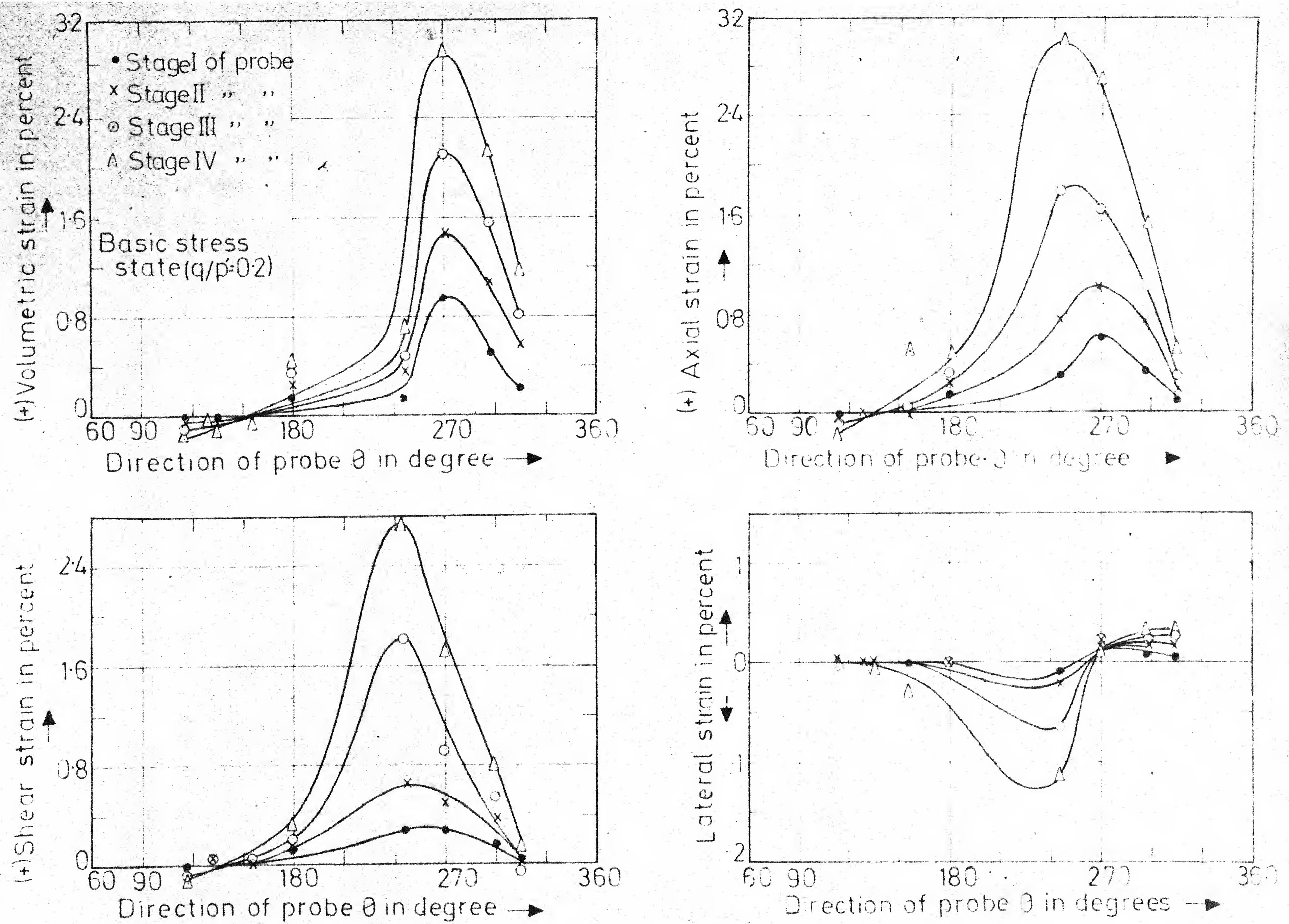


FIG. 4.12 MEASURED STRAINS

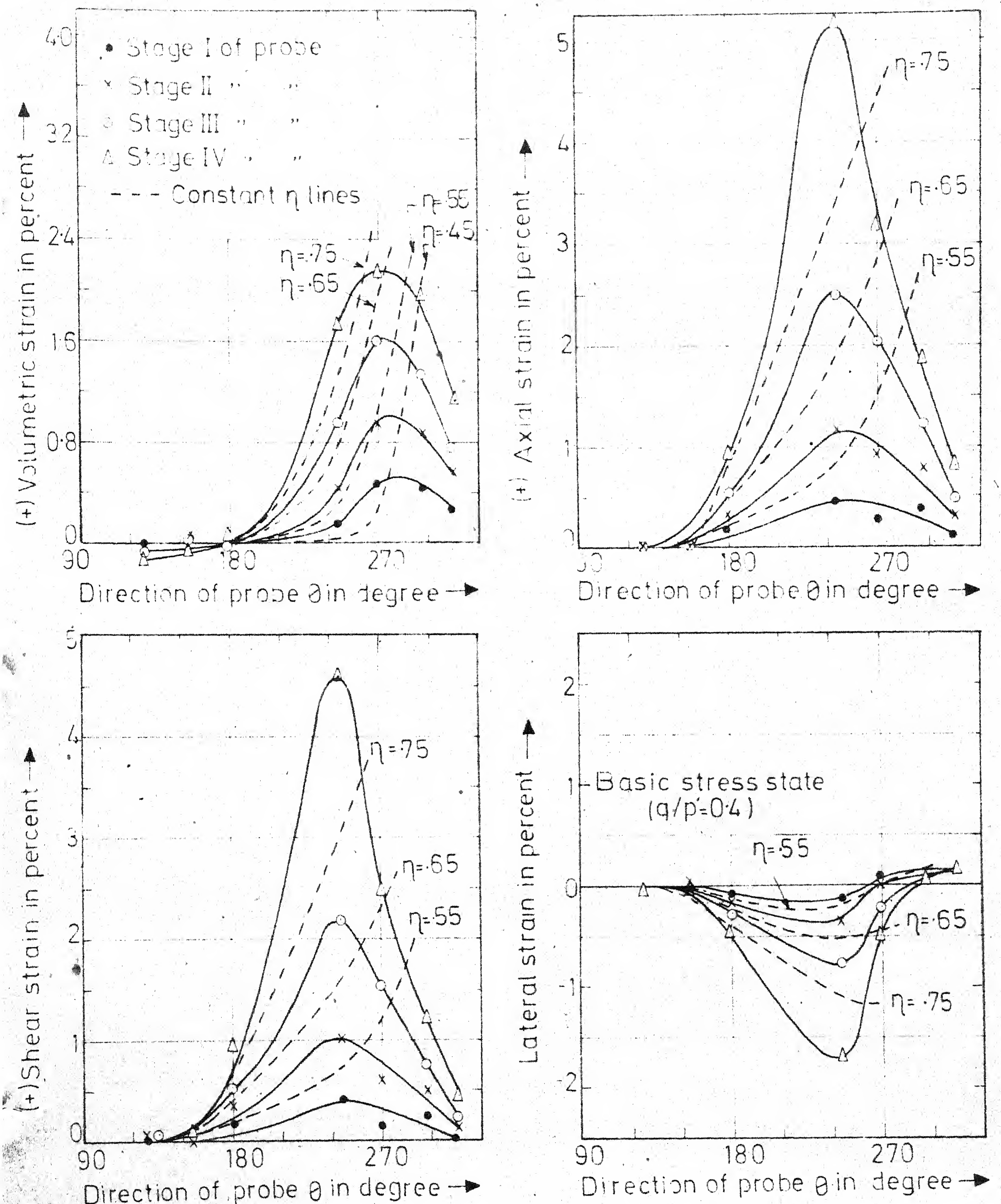


FIG 4·13 MEASURED STRAINS

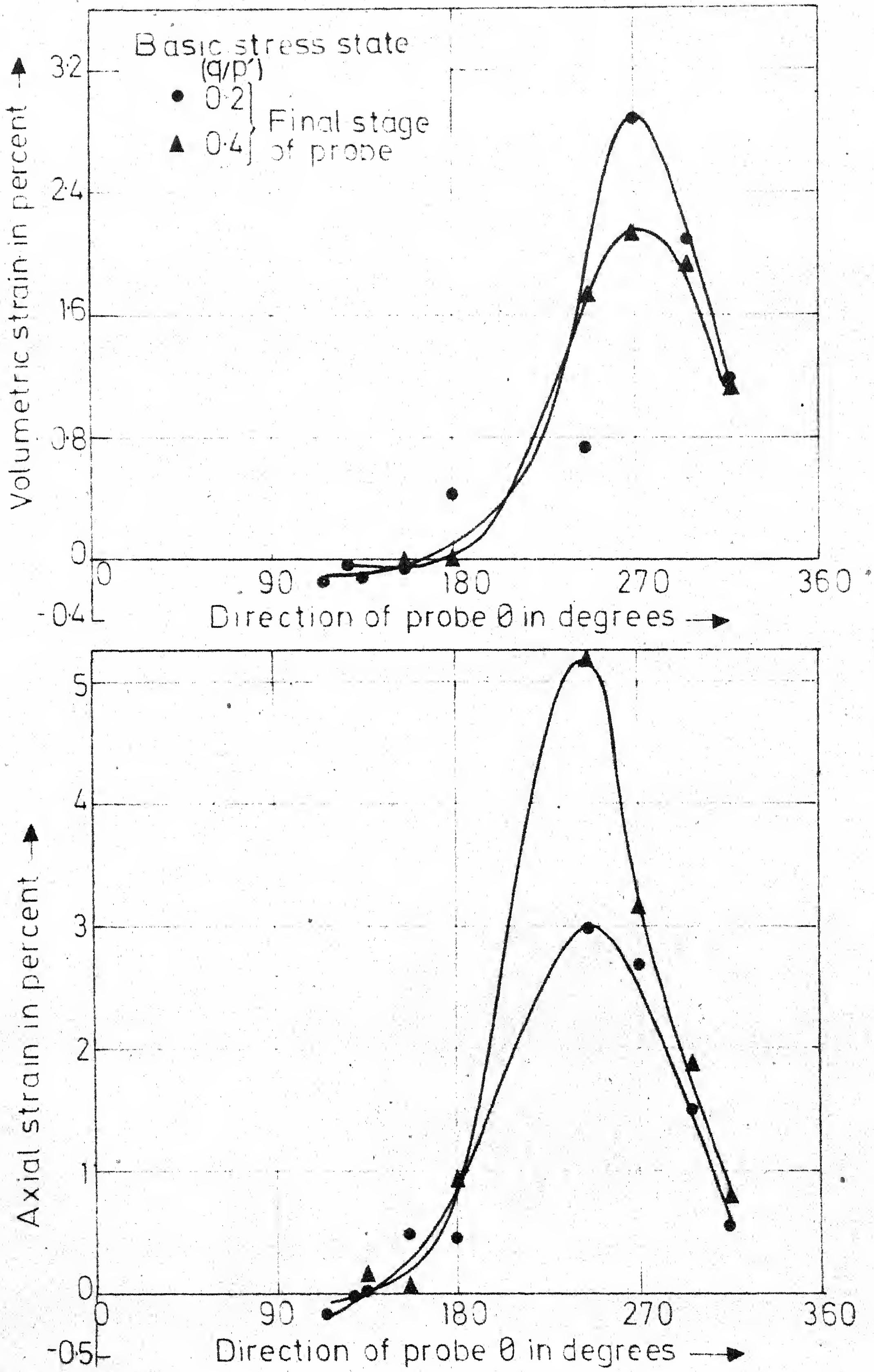
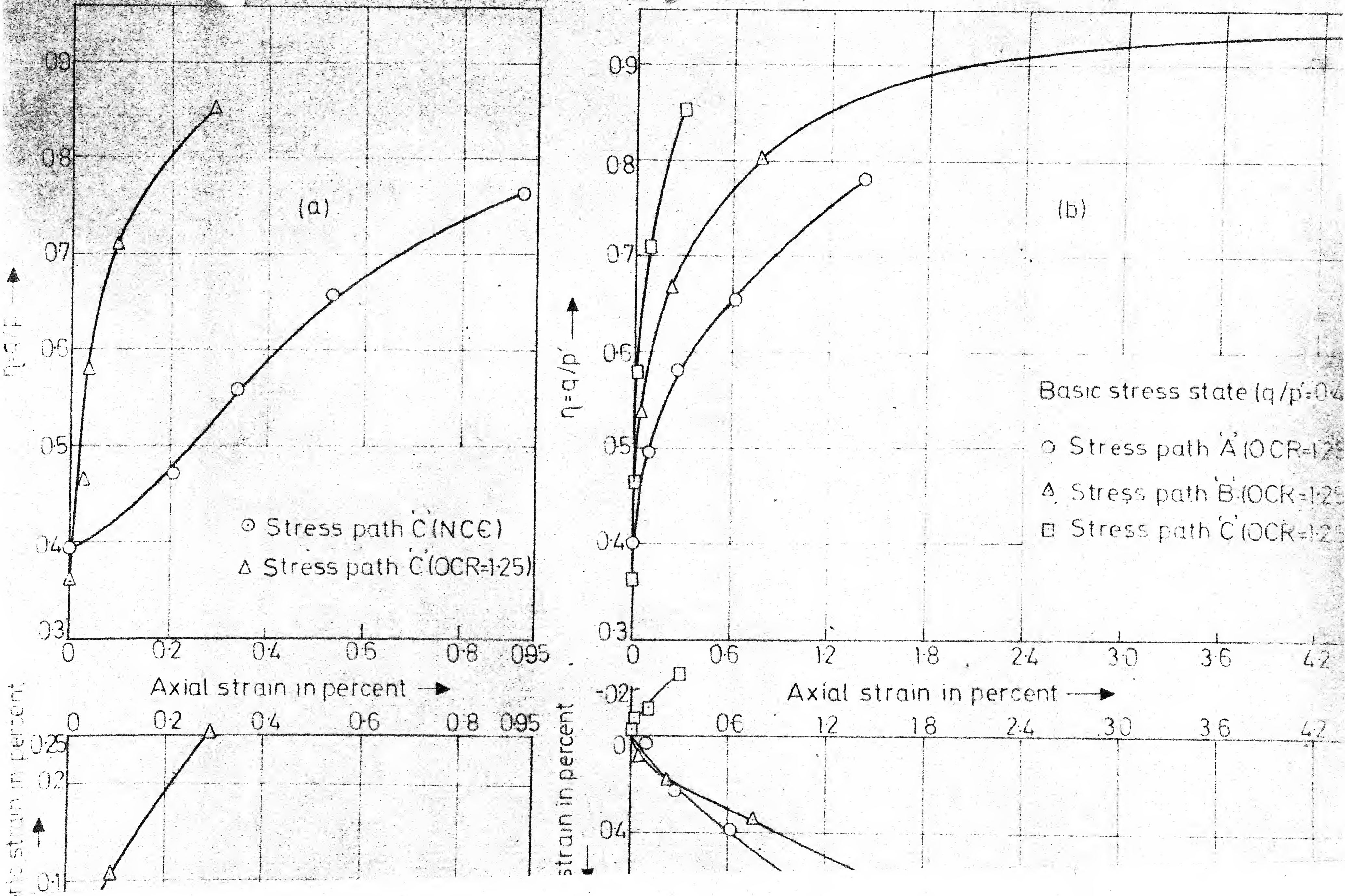


FIG. 4.14 MEASURED STRAINS FOR FINAL STAGE OF PROBES FOR BOTH THE BASIC STRESS STATES



(a)
⊙ Stress path C′ (NCE)
Δ Stress path C′ (OCR=1·25)
Axial strain in percent
(b)
Basic stress state (q/p′=0·4
○ Stress path A′ (OCR=1·2
Δ Stress path B′ (OCR=1·2
□ Stress path C′ (OCR=1·2
η=q/p′
Axial strain in percent
strain in percent

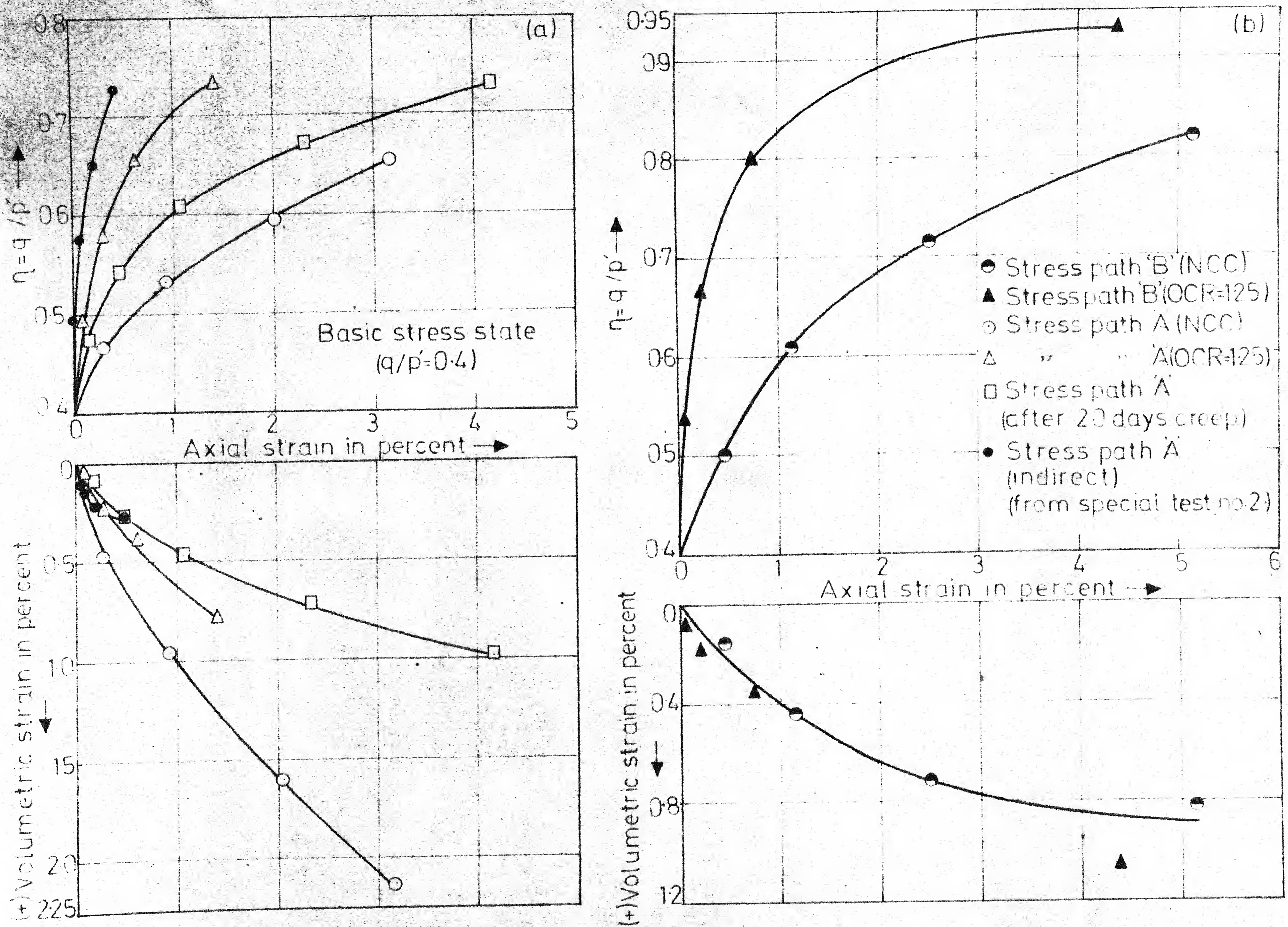


FIG. 4.16 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR OF LIGHTLY OVERCONSOLIDATED CLAY

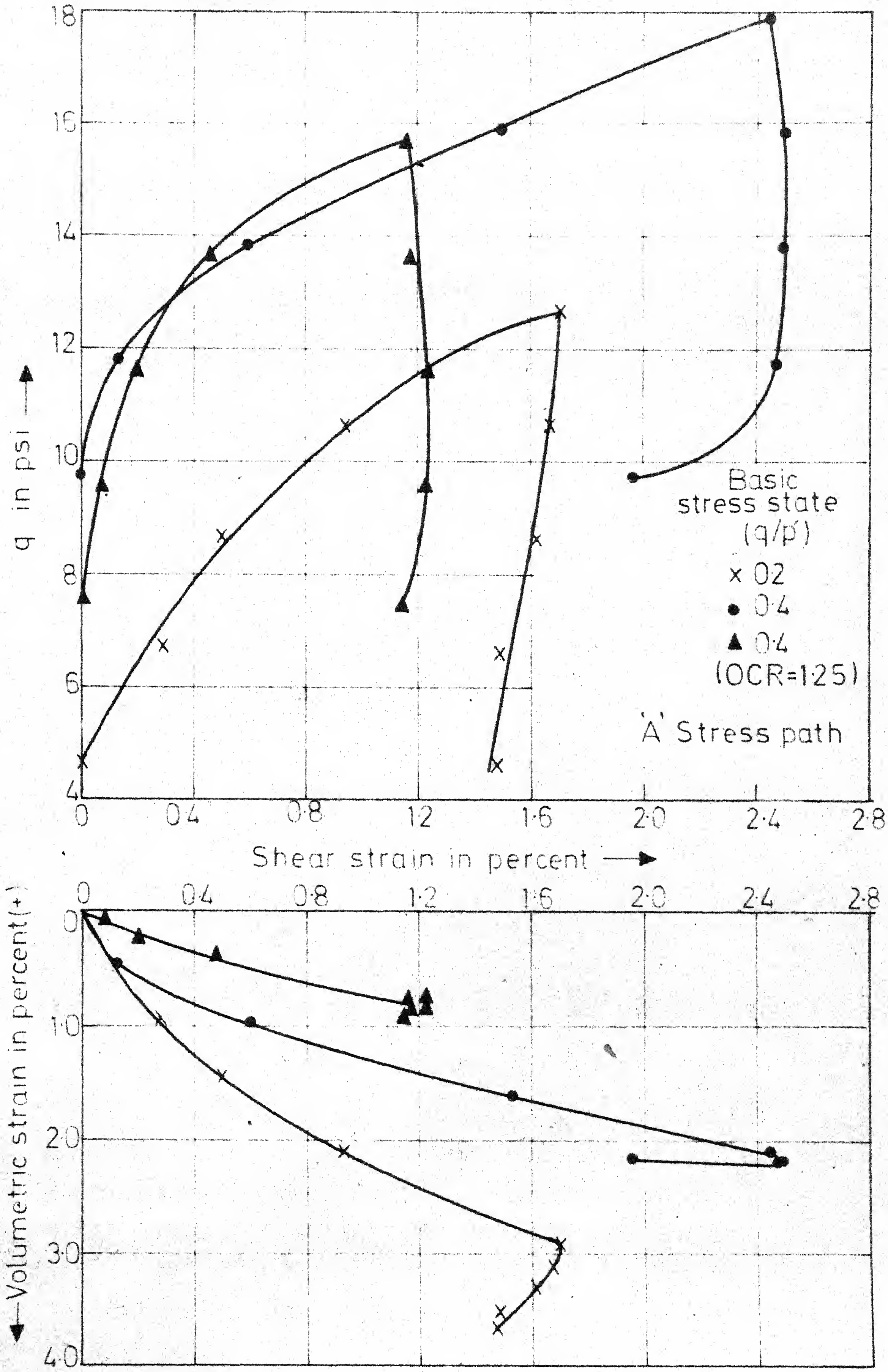


FIG. 4.17 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG 'A' STRESS PATH

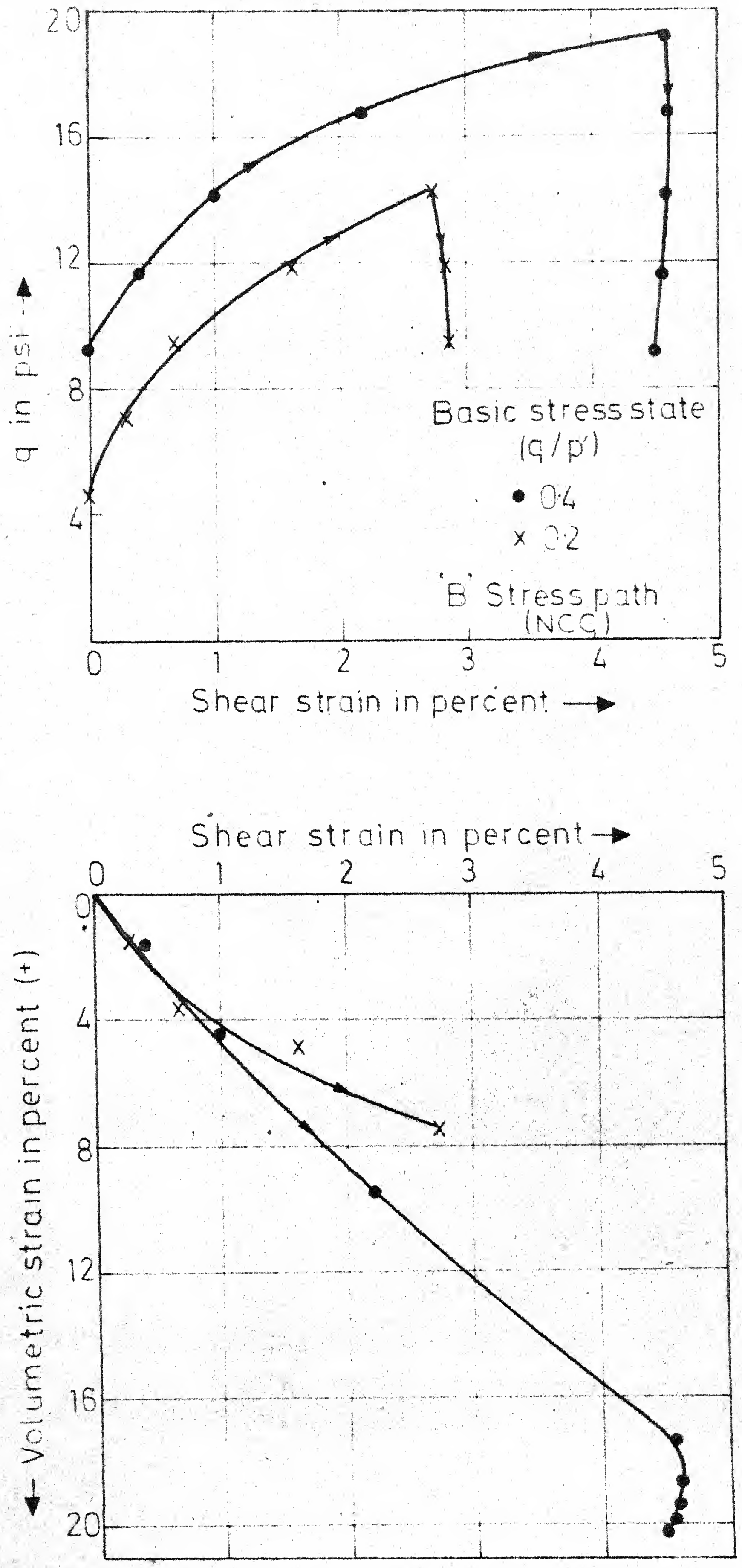


FIG. 4.18 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG 'B' STRESS PATH

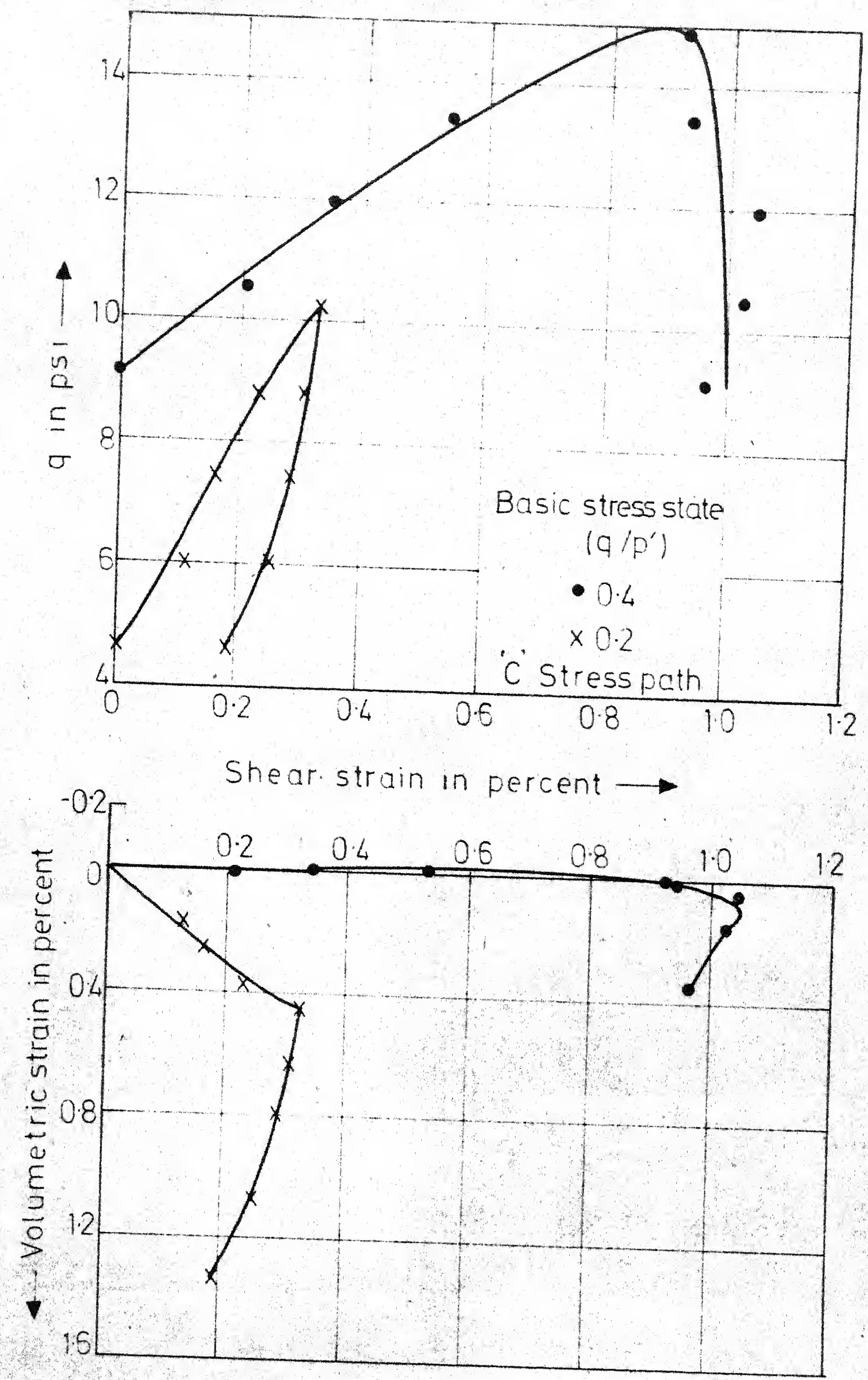


FIG 4.19 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG 'C' STRESS PATH

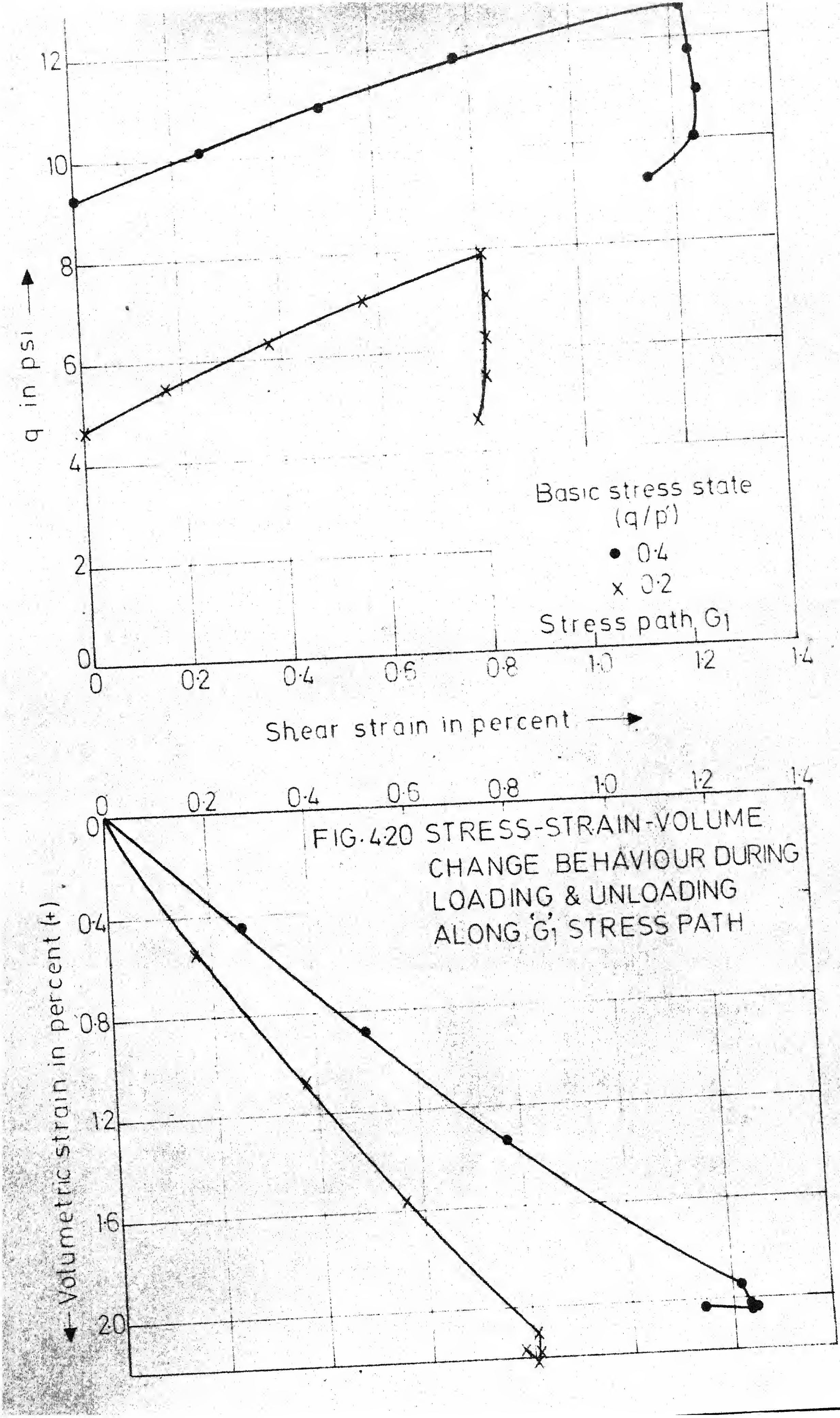
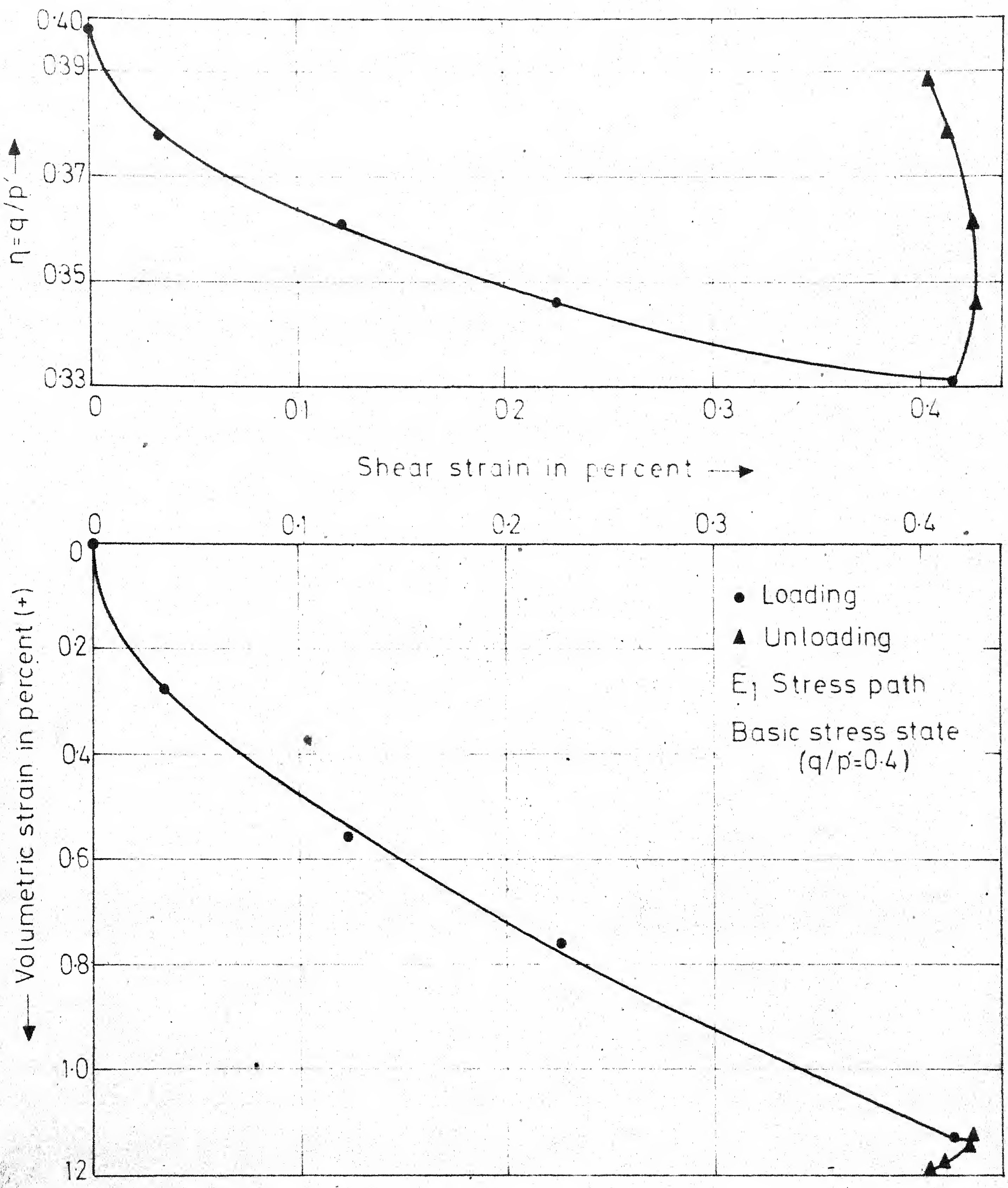
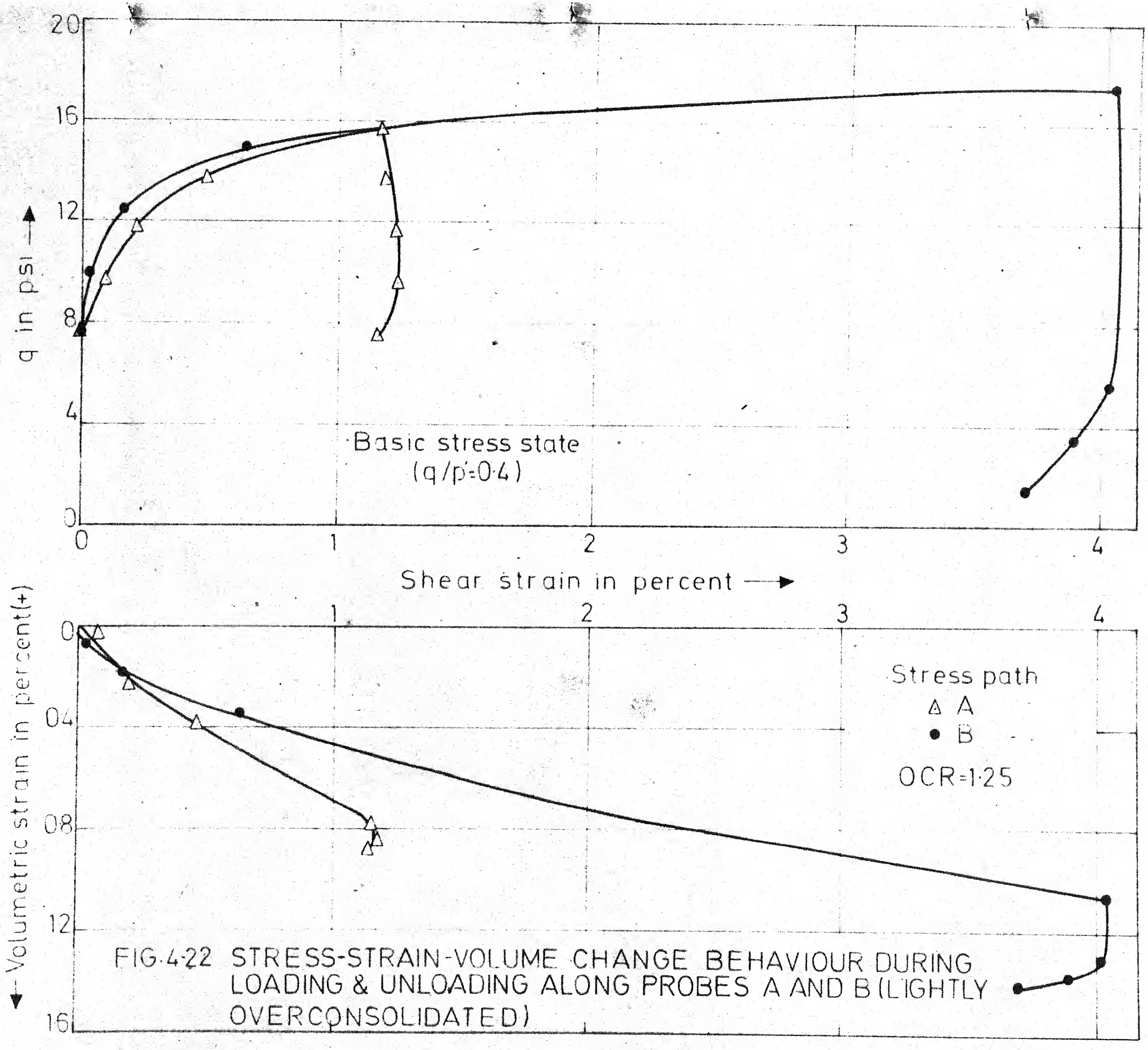


FIG. 4.20 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG 'G1' STRESS PATH



FIG·4·21 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG E_1 STRESS PATH



FIG·4·22 STRESS-STRAIN-VOLUME CHANGE BEHAVIOUR DURING LOADING & UNLOADING ALONG PROBES A AND B (LIGHTLY OVERCONSOLIDATED)

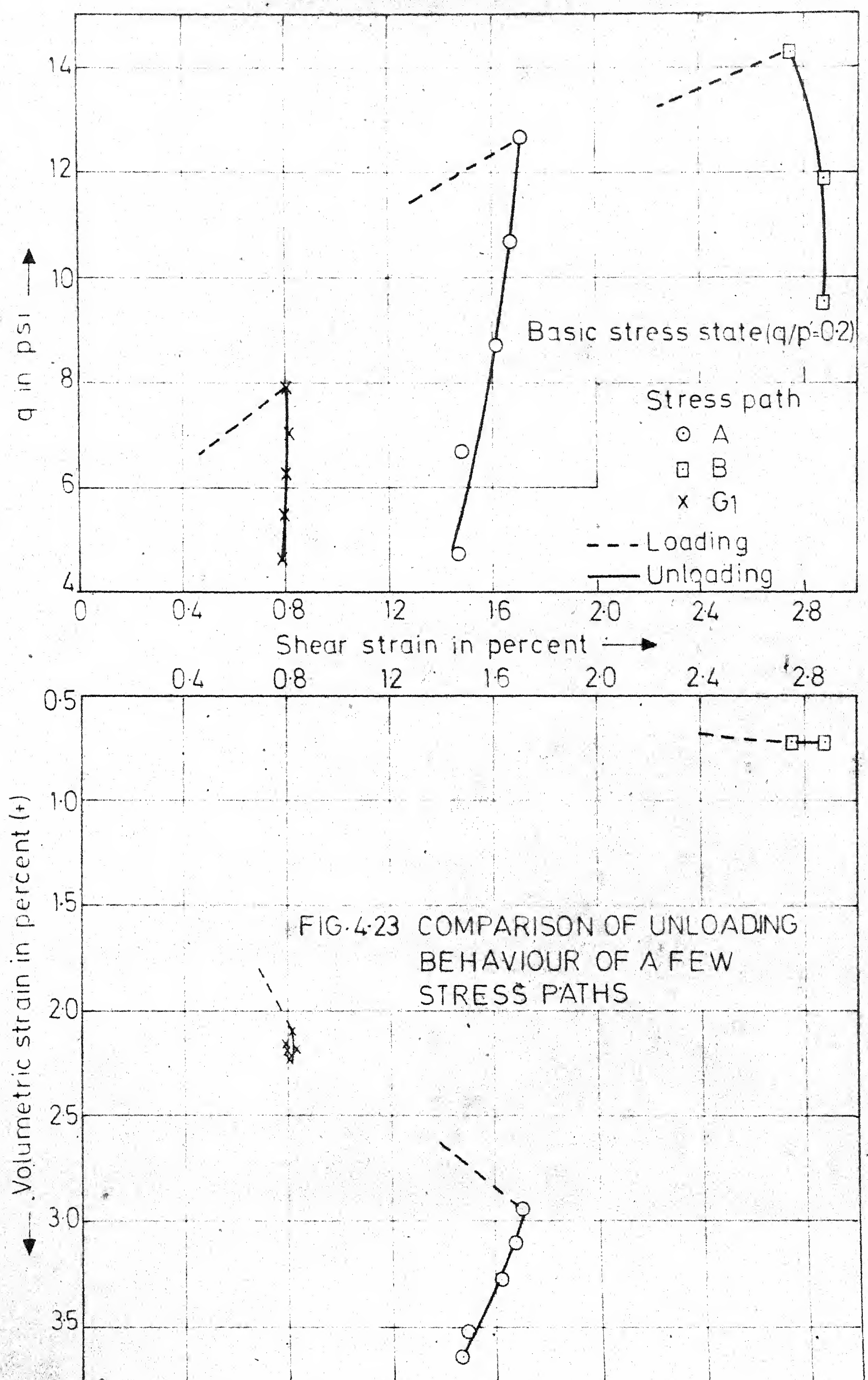


FIG. 4.23 COMPARISON OF UNLOADING BEHAVIOUR OF A FEW STRESS PATHS

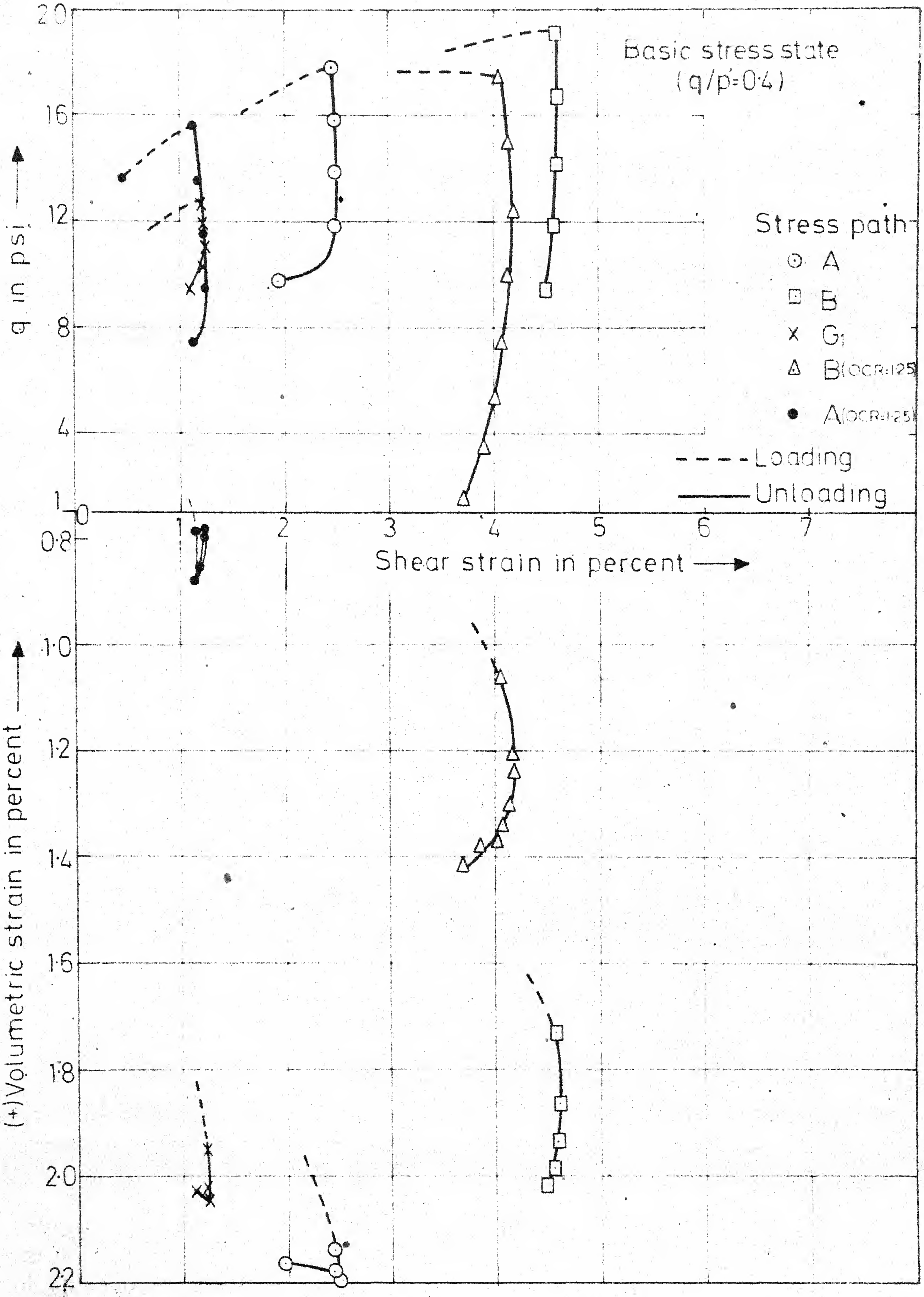


FIG. 4·24 COMPARISON OF UNLOADING BEHAVIOUR OF A FEW STRESS PATHS

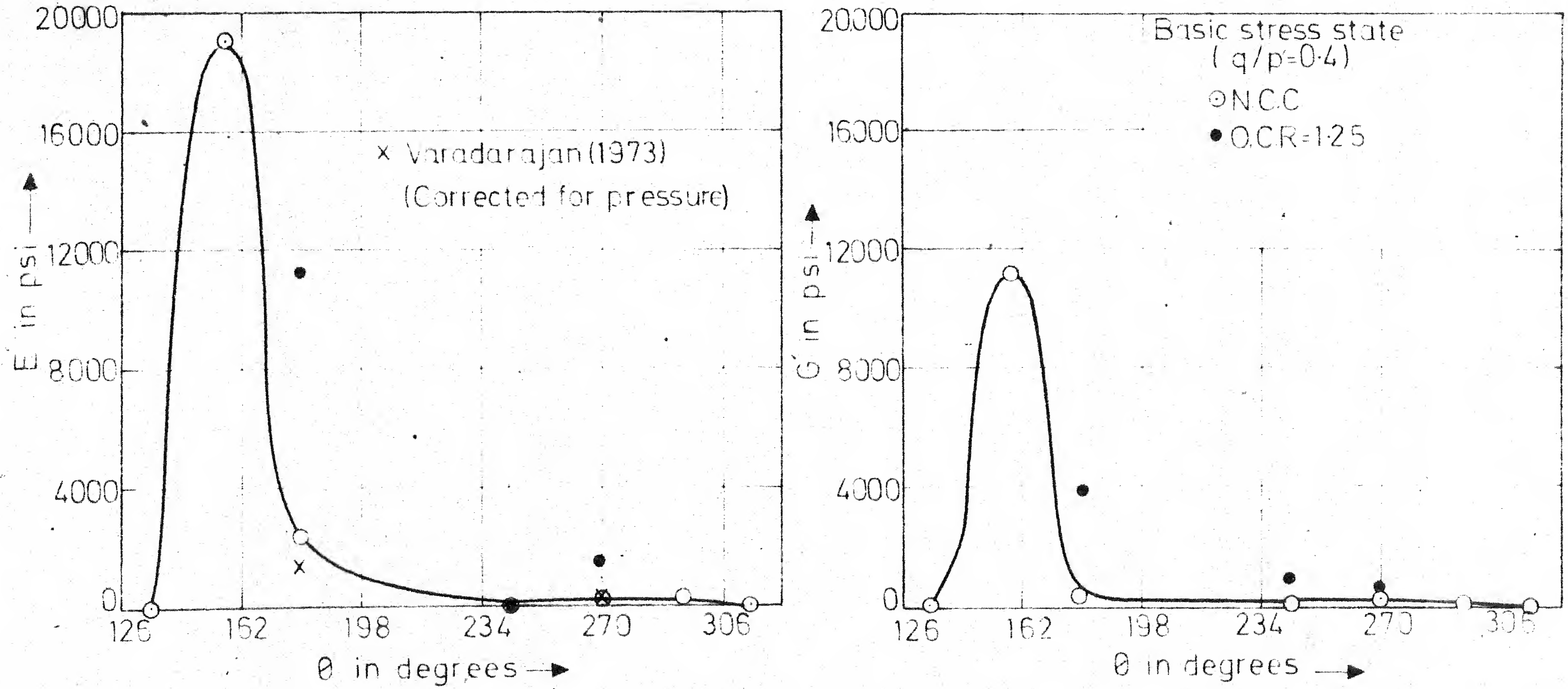


FIG. 4.25 VARIATION OF E' & G' WITH STRESS PATH (θ) AS PER THEORY OF ELASTICITY

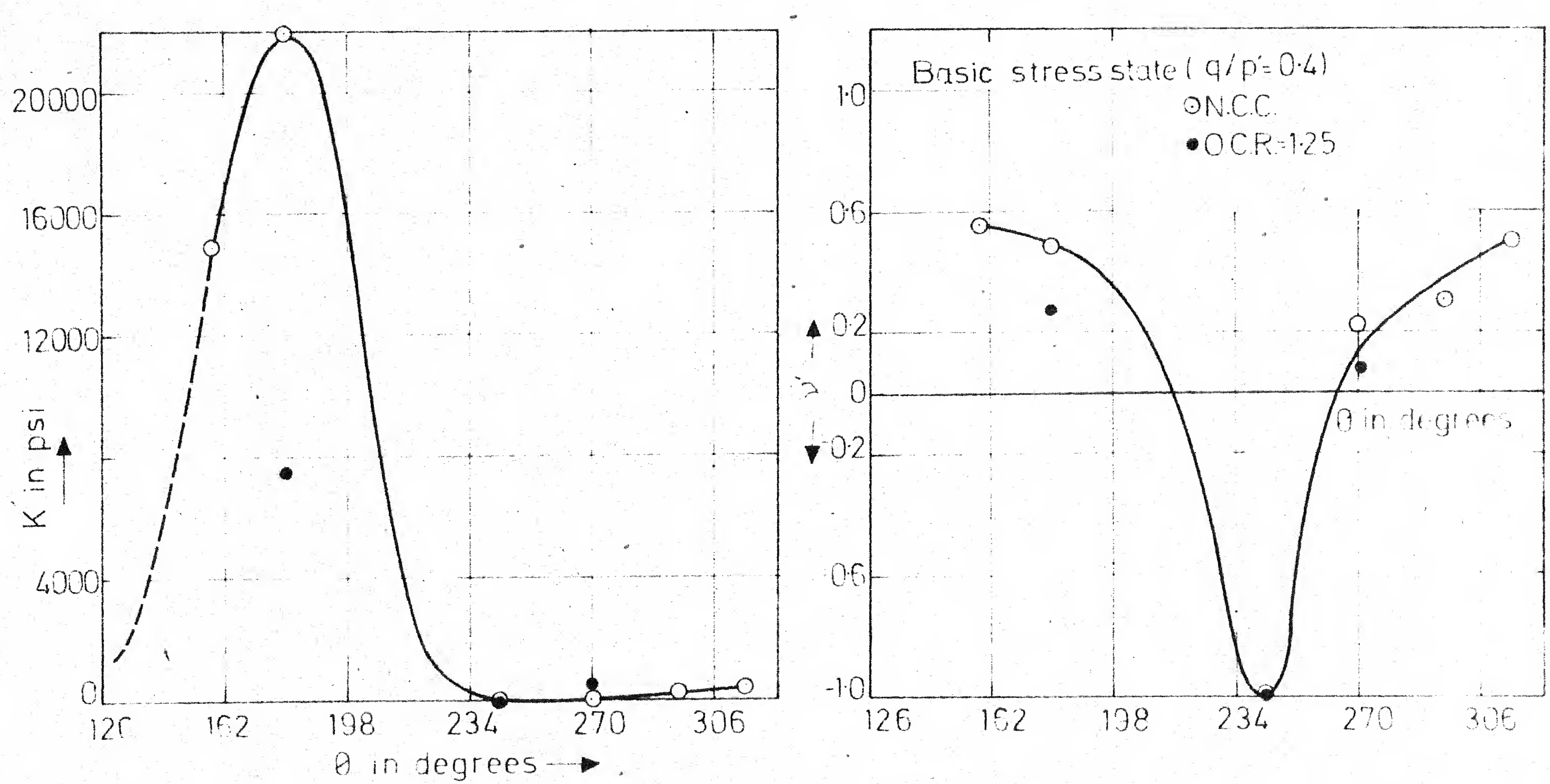


FIG.4.26 VARIATION OF K′ & ν′ WITH STRESS PATH (θ) AS PER THEORY OF ELASTICITY

CHAPTER 5

DRAINED CREEP BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED CLAY

5.1 GENERAL

Time readings for all the incremental creep tests along various stress paths were recorded for twenty four hours except for the anisotropic consolidation test along the basic stress state of $q/p' = 0.4$, for which the readings were recorded for a fortnight. In this chapter the drained creep behaviour for different stress paths has been presented in the form of percent strain (volumetric, shear and axial) variation with logarithm of time (minutes) both for normally and lightly overconsolidated samples. All the tests had the same starting pressure of 23.3 psi except for those starting from the lightly overconsolidated state. This study is mainly concerned with the "incremental" creep as opposed to the "prior" creep due to the anisotropic consolidation stress history of the clay. The slopes of the linear portion of percent strain versus log time relationship (logarithmic creep rate), referred to hereafter simply as "creep rates",have been designated as $c_{\alpha v}$, $c_{\alpha \varepsilon}$ and $c_{\alpha \varepsilon_1}$ for volumetric, shear and axial creep rates respectively. The variation of the creep rate with the stress ratio (η) could not be presented upto failure as the probes used in this study were small. However, one test (conventional drained i.e., 'A' stress path)

starting from the basic stress state of $q/p' = 0.1$ was conducted upto failure and this test has been discussed in detail along with the others.

5.2 DRAINED CREEP BEHAVIOUR OF THE ONE DIMENSIONAL CONSOLIDATION TEST

The anisotropic consolidation test along the basic stress state of $q/p' = 0.4$ (approximately k_o) was carried out to have an idea of the "prior" creep rate of the clay. Fig. 5.1 shows the axial strain variation with log time for the last stage of the test. The figure indicates a linear variation of axial strain with log time after the initial pore pressure dissipation. The slope of the linear portion ($c_{\alpha\varepsilon_1}$) is 0.6 and almost remains constant upto the time period of 16,000 minutes.

Anisotropic consolidation tests by other workers indicate that there is not much variation in c_α for smaller stress ratios (upto k_o) beyond which the creep rates increase with η [Ladd and Preston (1965), Barden (1969)]. Newland's (1973) results indicate that the shear creep rate ($c_{\alpha\varepsilon}$) is seen to increase rapidly with η. This is shown in Fig. 5.30(b). Yamanouchi & Yasuhara's (1975) tests indicate a linear variation of $c_{\alpha\gamma}$ and $c_{\alpha\varepsilon}$ with η. Clays subjected to k_o-consolidation before shearing show a little larger creep rate than the corresponding isotropically consolidated clays [Yamanouchi & Yasuhara (1975)]. Also the creep rates as observed in a k_o-triaxial test are generally larger than the corresponding rates in an

oedometer test, due to the side friction in the oedometer. One dimensional creep (oedometer) has been the subject of study by many workers. An excellent review of the factors affecting c_α is given by Mesri (1973).

5.3 DRAINED CREEP BEHAVIOUR OF THE NORMALLY CONSOLIDATED CLAY UNDER NON-k_o CONDITIONS

Test data from different basic stress states is presented first and then they are discussed.

5.3.1 Creep Test from the Basic Stress State of $q/p' = 0.1$.

For the conventional drained ('A' stress path) test from the basic stress state of $q/p' = 0.1$ conducted upto failure, variation of volumetric, shear and axial strains for some increments are shown against log time in Figs. 5.2 through 5.4. It is apparent from these curves that after a few hundred minutes of primary consolidation, a linear variation of strain with log time is obtained. $c_{\alpha v}$ (Fig. 5.2) appears to remain constant with stress ratio upto a certain stage beyond which it starts decreasing. $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ (respectively shown in Figs. 5.3 & 5.4) show an increasing trend with η . A detailed discussion of the variation of c_α with η is given later in this chapter.

5.3.2 Creep Tests along various Stress Paths from the Basic Stress State of $q/p' = 0.2$.

Figs. 5.5 through 5.8 show the variation of volumetric and axial strains for stress paths A, B, C and G_1. For E_1 stress

path, as shown in Fig. 5.15(b), only volumetric strain variation with log time is shown as the axial strains were extremely small. There does not appear to be much variation in $c_{\alpha v}$ with the stress ratio for a given stress path, although there is an evidence of the dependence of $c_{\alpha v}$ on stress path. $c_{\alpha\varepsilon_1}$ increases linearly with η . Stress path & stress ratio both affect $c_{\alpha\varepsilon_1}$. The results for this basic stress state are similar to those from the basic stress state of $q/p' = 0.4$, which are presented in detail in the next section.

5.3.3 Creep Tests along various Stress Paths from the Basic Stress State of $q/p' = 0.4$.

As this basic stress state happens to be near the k_o for this clay, it will be interesting to compare the results of various stress paths conducted from this state.

Figs. 5.9 and 5.10 give results of stress path A test. $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ are larger than $c_{\alpha v}$. There is some scatter in the results for $c_{\alpha v}$, but essentially it remains constant with the stress ratio, whereas $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ show an increase with η . Comparison of Fig. 5.11 with Fig. 5.10 shows that $c_{\alpha\varepsilon_1}$ values for stress path B test are always greater than the corresponding values for stress path A for the same stress ratio, clearly showing that the stress path affects the creep rates. As B stress path is the pure shear test (where p' remains constant), it shows higher shear and axial creep rates than the volumetric creep rate. For this stress path also, $c_{\alpha\varepsilon_1}$ increases linearly with η. Fig. 5.12 shows that for C

stress path, the trend is similar to A & B stress paths, although magnitudewise, creep rates are smaller for C than A & B stress paths for the same stress ratio, thus demonstrating clearly that the stress path has a significant influence over creep rates. $c_{\alpha v}$ is almost zero for the stress path C.

Figs. 5.13 & 5.14 indicate that for the typical footing stress path G_1 along which η variation is not substantial, $c_{\alpha v}$, $c_{\alpha \varepsilon}$ & $c_{\alpha \varepsilon_1}$ remain constant. However, from the results, it is clear that for stress path G_1 at higher stress ratios, $c_{\alpha \varepsilon}$ and $c_{\alpha \varepsilon_1}$ would show an increase with η. E_1 test (q-constant) [Fig. 5.15 & 5.16], a typical footing stress path, gives $c_{\alpha v}$ larger than the corresponding $c_{\alpha \varepsilon}$ & $c_{\alpha \varepsilon_1}$. This is quite consistent with the fact that a consolidation test would give a larger volumetric strain than the axial strain for a given increment. B stress path shows the opposite trend, being a pure shear test.

Figs. 5.17 and 5.18 show the results of F_1 stress path, which forms a continuation of the basic stress state. This is a part of special test 1 illustrated in Fig. 4.7 (chapter 4). As F_1 test had a different history and also due to the small increment, it gives smaller creep rates (actually it behaves as if the test was conducted from a lightly overconsolidated state) [Bishop & Lovenbury (1969), Mesri (1973), Edgers et. al. (1973)]. This is clear from the magnitudes of $c_{\alpha v}$, $c_{\alpha \varepsilon}$ & $c_{\alpha \varepsilon_1}$, which are 0.29, 0.152 & 0.285 respectively. $c_{\alpha \varepsilon_1}$ from this test is almost half as compared to the regular

anisotropic test value (Fig. 5.1). Hence the anisotropic consolidation tests would indicate different creep rates for normally & lightly overconsolidated samples.

5.4 DRAINED CREEP BEHAVIOUR OF THE LIGHTLY OVERCONSOLIDATED CLAY (OCR = 1.25) UNDER NON-k_o CONDITION

Test results of stress paths A and B from an OCR of 1.25 are described along with the results of the special conventional drained test conducted on a sample which was allowed 20 day "prior" creep before shearing. Figs. 5.19 through 5.21 show the variation of volumetric, shear and axial strains with log time for stress path A; Figs. 5.22 through 5.24 give these variations for stress path B; whereas Figs. 5.25 through 5.27 show the variations for the special test (A stress path) with 20 day "prior" creep.

The strain-log time curves for lightly overconsolidated samples are similar in shape to those obtained for the normally consolidated samples. Tables 5.1 through 5.5 list the values of creep rates for various stages of the tests. It is seen from the tables that $\bar{c}_{\alpha\varepsilon_1}$ values for stress path A, corresponding to a stress ratio of 0.65 are 1.23, 0.28 and 0.32 respectively for normally, lightly overconsolidated clay samples and the special test. Similarly $c_{\alpha v}$ corresponding to η of 0.58 are 0.6, 0.21 and 0.1 respectively for the three cases. These results indicate that in general the creep rates are considerably smaller for the lightly overconsolidated samples as compared to the rates for the normally consolidated samples.

There is a small scatter in $c_{\alpha v}$, but the trend is essentially constant with η. These results along with those on normally consolidated clay will now be discussed in detail.

5.5. DISCUSSION OF RESULTS

The drained creep test results clearly show a consistent pattern in that after the excess pore pressures associated with primary consolidation dissipate, strains (volumetric, shear & axial) vary linearly with log time. The curves for all stress paths are similar to the type III curve of Lo (1961) due to the small increment ratio used. The linear variation of strain with log time has been observed by many workers also on various soils. Anisotropic consolidation test along k_o (basic stress state $q/p' = 0.4$) indicated that the slope c_α remains constant with time (upto a fortnight). However, long term tests described by Lo (1961) (oedometer), Bishop and Lovenbury (1969) (both oedometer and drained triaxial) on undisturbed samples indicate that this linear variation is observed for only 60 to 200 days depending upon the clay and later on "instabilities" developed, which changed altogether this simple behaviour. As has been rightly pointed out by Edgers et. al. (1973) and Mesri (1973) that these "instabilities" could result from experimental difficulties, such as temperature changes, external vibration, physico-chemical changes in the sample due to cementation or bacterial growth at the particular laboratory temperature etc., although very specialized equipment for these tests was developed by Bishop and Lovenbury. These "instabilities", which

they term as fundamental to insitu soil behaviour, would render inadequate the many simple logarithmic or power laws used to present the time deformation behaviour of soils. Bishop & Lovenbury's data also indicates that axial strain rate ($c_{\alpha\varepsilon_1}$) is not constant but varies slightly nonlinearly with time. Bjerrum (1967) reports creep rates decreasing with time. Hence, there is a great need of accurate long term insitu creep data. However, laboratory tests on uniform & similar samples have their own merit in a fundamental study of this type.

A comparison of test results from the two basic stress states of $q/p' = 0.2$ and 0.4 indicates that barring a few cases, in general, the creep rates are higher for tests starting from the lower basic stress state. This, of course, may not be true for higher stress ratios. As the basic stress state of $q/p' = 0.4$ represents approximately the k_o of the clay, its results would be mainly discussed, as they would be more relevant to a field problem (similar explanation would be valid for tests from the other basic stress state).

The test results indicate that the incremental creep rate is very much a function of the stress path and the stress ratio. For a given stress ratio, creep rates are different for different stress paths. This is indicated in Fig. 5.28, which represents the variation of the axial creep rate ($c_{\alpha\varepsilon_1}$) with the

direction of the stress path θ(θ has been defined in Fig. 3.3). The solid curve in this figure has been drawn through the points that correspond to creep rates as obtained at the end of the final stage of each stress path test. Dotted curves in the figure represent constant stress ratio (η) lines. These dotted lines run across a number of stress paths, indicating the dependence of axial creep rate on the stress path. In the range of θ shown in this figure, stress paths such as C, B, A, G_1, E_1 are encompassed. Constant η-lines of less than 0.55 would be relatively much flatter and thus would indicate that axial creep rates donot depend to a large extent on stress path for low stress ratios. Similar trend would be observed for shear creep rates ($c_{\alpha\varepsilon}$) when plotted in this form. Murayama and Shibata (1964) have clearly shown that for stress path E_1, $c_{\alpha\varepsilon_1}$ virtually remains constant with pressure and since q is constant for E_1 stress path, it would indicate that $c_{\alpha\varepsilon_1}$ is almost independent of η for this stress path. This is shown in Fig. 5.29(a). They have indicated considerable difference in results of stress paths A and E_1. Newland's (1973) creep data reproduced in Fig. 5.30(b) for stress path E_1 indicates that shear creep rate ($c_{\alpha\varepsilon}$) stays almost constant with η. Burland (1969) has indicated tnat $c_{\alpha v}$ for E_1 stress path is larger than the corresponding $c_{\alpha\varepsilon}$. The present investigation substantiates the findings of these authors. As explained earlier, the fact that volumetric creep rates are found to be higher than the corresponding shear creep rates for this stress path is understandable, as this,

being a consolidation test (from an anisotropic line), would show higher volumetric strains than shear strains for the same increment. The reverse trend is obtained for stress path B, which is a pure shear test (Refer Fig. 5.6).

From Bishop and Lovenbury (1969) data, it can be concluded that for undisturbed London clay, $c_{\alpha\varepsilon_1}$ is much larger for stress path A than for C. For a stress level of 70-82 percent, $c_{\alpha\varepsilon_1}$ values for stress paths A & C are 0.17 and 0.06 respectively (slope taken in the range of 1 to 10 days), indicating that the ratio $\frac{c_{\alpha\varepsilon_1}(A)}{c_{\alpha\varepsilon_1}(C)}$ is about 3. This ratio is 4 for stress ratio η equal to 0.656 in this study. The only available data for A and C stress paths (although for an overconsolidated clay) confirm the findings of this investigation that creep rates for stress path C are relatively much smaller.

The test results for G_1 stress path indicate that for the probe length investigated (η does not vary much), $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ show an increase with η, although in the beginning it stays constant. $c_{\alpha v}$, however, remains constant. Poulos et. al's (1975) recent study of G_1 stress path, in their effort to predict the creep rates by the generalized stress path method [as originally indicated by Koizumi and Ito (1964) and Barden (1969)] have indicated that aging does not result in a decrease in $c_{\alpha\varepsilon_1}$. Fig. 5.31 shows the results of Poulos et. al. The fact that Poulos et. al.did not observe any change in $c_{\alpha\varepsilon_1}$ for an aged sample does not mean that this would be true for

other soils and stress paths. This may be typical of the stress path they have chosen, where the change in stresses is not significant to cause a substantial change in η. Again Kaolin as used by them has lower creep potential compared to more active clays. On the contrary the study on lightly overconsolidated samples and the results of the special conventional drained test on a 20 day aged sample, have clearly shown that there is a considerable reduction of creep rates for an aged sample for A and B stress paths. This is in order as for the normally consolidated clays where contact stresses are relatively high, c_α will be higher than for overconsolidated soils, where contact stresses are lower [Mesri (1973)].

The tests in this study have been conducted at the same pressure and for samples of 3 inch height. Pressure increment ratio ($\Delta p/p$) was also almost constant (small $\Delta p/p$). However, as has been studied in detail by many investigators [Barden (1969), Ladd and Preston (1965), Mesri (1973), Walker (1969), Yamanouchi & Yasuhara (1975)], c_α seems to remain unaffected by $\Delta p/p$ ratio, height of the sample & pressure. It can, therefore, be concluded that, in general, drained creep rates are very much affected by the stress path.

5.5.1 Variation of Creep Rates With Stress Ratio (η).

Barden (1969) mentions that if shear deformation including failure is governed by the stress ratio, it is logical to expect creep deformations also to be affected by it. Test results presented in

this chapter have clearly indicated that a knowledge of deviator stress alone is not sufficient for analysing the drained creep behaviour. Stress ratio η and the stress path are the two main factors which affect creep rates.

Figures 5.32(b), 5.33 and 5.34(b) give the variation of $c_{\alpha\varepsilon}$, $c_{\alpha\varepsilon_1}$ with η mainly for stress path A and B tests (for normally and lightly overconsolidated samples). Test results for other stress paths have also been shown for comparison. Figs. 5.32(b) and 5.34(b) indicate that the variation is similar for both normally and lightly overconsolidated samples, i.e., $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ increase linearly with the stress ratio (η) upto an "yield value", after which it increases very rapidly with stress level. At stresses higher than the yield value, the drained creep developed into creep rupture. This behaviour is typical for stress paths A and B. Although points for stress paths G_1, F_1, E_1 are also shown on these figures for comparison, it should not be assumed that these stress paths would exhibit similar behaviour. G_1 and E_1 stress paths have been discussed earlier and it was shown that for E_1 stress path; $c_{\alpha\varepsilon}$, $c_{\alpha\varepsilon_1}$ remain constant. For stress path G_1, at higher stress ratios, $c_{\alpha\varepsilon}$ and $c_{\alpha\varepsilon_1}$ would indicate a linear variation with η. This linear variation, as observed for stress paths A, B, C & G_1 is typical of the drained creep behaviour observed by Barden (1969) & Walker (1969) for stress path A; Yamanouchi and Yasuhara (1975) for stress path B and Murayama and Shibata (1961, 1964) for stress path A. The "yield value" as observed in this investigation

for A and B stress paths confirms the findings of Murayama and Shibata (1961,1964). Figs. 5.35(a) and (b) respectively present the results of Walker (1969) and Yamanouchi & Yasuhara (1975).

This linear relationship can be represented as :

$$c_{\alpha\varepsilon} = \frac{d\varepsilon}{d(\log_{10} t)} \propto \eta = A_1 \eta \tag{5.1}$$

and $$c_{\alpha\varepsilon_1} = \frac{d\varepsilon_1}{d(\log_{10} t)} \propto \eta = A_2 \eta \tag{5.2}$$

where A_1 and A_2 are constants, giving the slope of the $c_{\alpha\varepsilon}(c_{\alpha\varepsilon_1})$ versus η diagrams.

The relations represented by equations (5.1) and (5.2) are typical for stress paths A and B and <u>cannot be said</u> to be general for drained creep behaviour along all stress paths (e.g. E_1 stress path) hitherto assumed by other workers. Further these relationships are valid only upto the yield value.

So far the variation of shear and axial creep rates with η has been discussed. Variation of $c_{\alpha v}$ (volumetric creep rate) with η will now be presented. Figs. 5.32(a) and 5.34(a) show respectively the behaviour observed for normally and lightly overconsolidated clays. Test results of stress paths A and B from the two basic stress states show that essentially $c_{\alpha v}$ remains constant with η upto a certain stress level (which seems to be near the "yield value"), beyond which $c_{\alpha v}$ starts decreasing and eventually shows a tendency towards zero

volumetric creep rate at failure. Points from other stress path tests are shown in the figure for comparison. There is a difference in the $c_{\alpha v}$ values for the two basic stress states shown in the figure. Although Walker (1969) got different $c_{\alpha v}$ values for tests in simple shear and triaxial apparatus, he indicated that isotropic consolidation and A stress path tests give same $c_{\alpha v}$. At present more data is needed to clarify this point. The scatter in $c_{\alpha v}$ values as observed by Walker; Newland; Yamanouchi and Yasuhara is of the same order as obtained in the present investigation. Some of the scatter in the figure is due to the differences in stress paths followed in the tests, indicating the dependence of $c_{\alpha v}$ on stress path. However, as indicated earlier, this sample ($q/p' = 0.1$) was obtained from a different slurry and as such the observed values of $c_{\alpha v}$ for this test may as well be due to the sample differences and for this reason alone a separate curve through these points has been shown in Fig. 5.32(a). It is difficult to obtain correct volumetric creep rates in triaxial due to the leakage past membranes & 'O' rings etc. in long duration tests. It can, therefore, be concluded with the scatter as observed, that $c_{\alpha v}$ remains constant with η for all stress paths. However, $c_{\alpha v}$ depends on the stress path. The tendency of $c_{\alpha v}$ to decrease beyond the "yield value" is quite relevant in view of the fact that normally consolidated clays undergo shear at a very small volume at failure. For lightly overconsolidated clay (though there is evidently some scatter), $c_{\alpha v}$ values are smaller as compared to the values of normally consolidated clay.

Available variations of $c_{\alpha v}$ with stress ratio (η) in the literature are by Walker (1969) [shown in Fig. 5.35(a)], Newland (1973) [shown in Fig. 5.30] and by Yamanouchi and Yasuhara (1975) [shown in Fig. 5.35 (b)]. The trend expected by Adachi and Okano (1974) is shown by dotted line along Walker's curve in Fig. 5.35 (a). Whereas the results shown by Newland (1973) give lot of scatter and mainly consist of E_1 tests, a careful look at his figure would indicate that an average vertical line is not at all a bad fit upto $\eta = 0.5$, beyond which $c_{\alpha v}$ follows the trend indicated by Newland. With this modification, the results obtained in this investigation are similar to Newland's. Walker's results as presented in Fig. 5.35(a) show a vertical line variation upto all stress levels. However, Walker's experimental points do indicate a variation similar to the one obtained by the author in this investigation. Adachi and Okano's (1974) dotted line variation shown in Fig. 5.35(a) (along with Walker's line) almost give the variation obtained in this investigation.

The straight line variation of $c_{\alpha v}$ with η shown by Yamanouchi and Yasuhara (1975) [Fig. 5.35(b)] for stress path B tests, does not appear consistent with the fact that as failure is approached, the volumetric creep should reduce. The dotted line shown in this figure, consistent with the interpretation made here, suggests that within the scatter range seen in other investigations, $c_{\alpha v}$ in this case (B stress path) also is independent of η.

It can, therefore, be concluded that volumetric creep rate ($c_{\alpha v}$) is independent of stress level (η) upto the "yield value". However, stress path does seem to affect $c_{\alpha v}$.

The observed variation upto the "yield value" can be written as:

$$c_{\alpha v} = \frac{dv}{d(\log_{10} t)} = A_3 = \text{constant} \tag{5.3}$$

Equations (5.1) through (5.3) corroborate in general the findings of Walker (1969).

5.6 CONCLUSIONS

The study carried out by the author on the drained creep behaviour along various stress paths is believed to be one of the first few detailed and systematic investigations. The curves, after an initial portion of a few hundred minutes, resolve into linear strain-log time plots both for normally and lightly overconsolidated samples for various stress paths. The creep rates (for A and B stress paths) observed for lightly overconsolidated clay are much smaller compared to the corresponding values for the normally consolidated clay. The special conventional drained test for a twenty day aged sample showed similar behaviour. This is contrary to the findings of Poulos et. al. (1975) who didnot observe any change in the creep rates for aged samples. This, however, appears to be due to the particular stress path considered (G_1)

which does not show much variation in η and the low creep potential of Kaolin used.

The results clearly indicate that the drained creep rates depend very much on the stress path. For the same ratio, the variation in creep rates for different stress paths is very significant (Fig. 5.28). This proves and justifies the statements of Barden (1969) and Yamanouchi & Yasuhara (1975), who have presented results of A and B stress paths respectively and have made these general observations.

Results also substantiate the findings of Walker (1969) that the shear creep rate is linearly related to η. However, as has been pointed out in the discussions that this linear variation is upto an "yield value", beyond which the axial and shear creep rates become excessive leading to creep rupture at very high η-values. The influence of stress level on the creep behaviour of soils is better documented than the influence of any other variable.
It appears that creep rupture will develop at lower stress levels in undrained creep than in drained creep because of the induced pore pressures. The linear variation of $c_{\alpha\varepsilon}$ & $c_{\alpha\varepsilon_1}$ with η cannot be generalized as a characteristic feature of the drained creep behaviour, as it is dependent on the stress path e.g. for E_1 stress path, creep rates are independent of η.

Volumetric creep rates ($c_{\alpha v}$) are independent of η upto the "yield value", beyond which a decrease in $c_{\alpha v}$ occurs. Different workers have put forward different view-points on this aspect. However, it is felt by the author that the variation of $c_{\alpha v}$ as presented, appears to be the most likely trend. Yamanouchi & Yasuhara's linear variation is not compatible with the known feature of clays to undergo shear at almost zero volumetric creep rate. Volumetric creep rates, with the limited data available in this study, suggest path dependence, although further detailed investigation is required to clarify this point.

In the end it is pointed out that due to the dependence of creep rates on the stress path, as observed in this study, a realistic idea of the deformations could be had for any problem by predicting the creep rates of various soil elements, depending upon the stress path in a given problem. A model to predict the creep rates for different stress paths from the results of two known tests is described in Chapter 6. Also the usefulness of the Berkeley rate process model to predict the drained creep behaviour is discussed in the next chapter.

TABLE 5.1

Creep Rates for 'A' Stress path (for N.C.C.)

η	Creep rates		
	$c_{\alpha v}$	$c_{\alpha\varepsilon}$	$c_{\alpha\varepsilon_1}$
.468	.42	-	-
.534	.485	-	-
.595	.605	.76	1.05
.656	.415	.84	1.23

TABLE 5.2

Creep Rates for 'A' Stress Path for Lightly Overconsolidated Clay (OCR = 1.25)

η	Creep rates		
	$c_{\alpha v}$	$c_{\alpha\varepsilon}$	$c_{\alpha\varepsilon_1}$
.494	-	-	.048
.58	.211	.074	.14
.655	-	-	.28
.732	.515	.675	.91

TABLE 5.3

Creep Rates for 'A' Stress Path for the Special Test (with 20 day "prior" creep)

η	Creep rates		
	$c_{\alpha v}$	$c_{\alpha\varepsilon}$	$c_{\alpha\varepsilon_1}$
.585	.09	.07	.08
.647	.163	.295	.32
.71	.158	.46	.53
.765	.228	1.22	1.36
.82	-	-	1.82

TABLE 5.4

Axial Creep Rates for 'B' Stress Path
(for N.C.C.)

η	$c_{\alpha\varepsilon_1}$
.502	.43
.61	.80
.715	1.12
.82	2.09

TABLE 5.5

Axial Creep Rates for 'B' Stress Path
for Lightly Overconsolidated Clay
(OCR = 1.25).

η	$c_{\alpha\varepsilon_1}$
.534	.05
.667	.145
.80	.49
.931	3.75

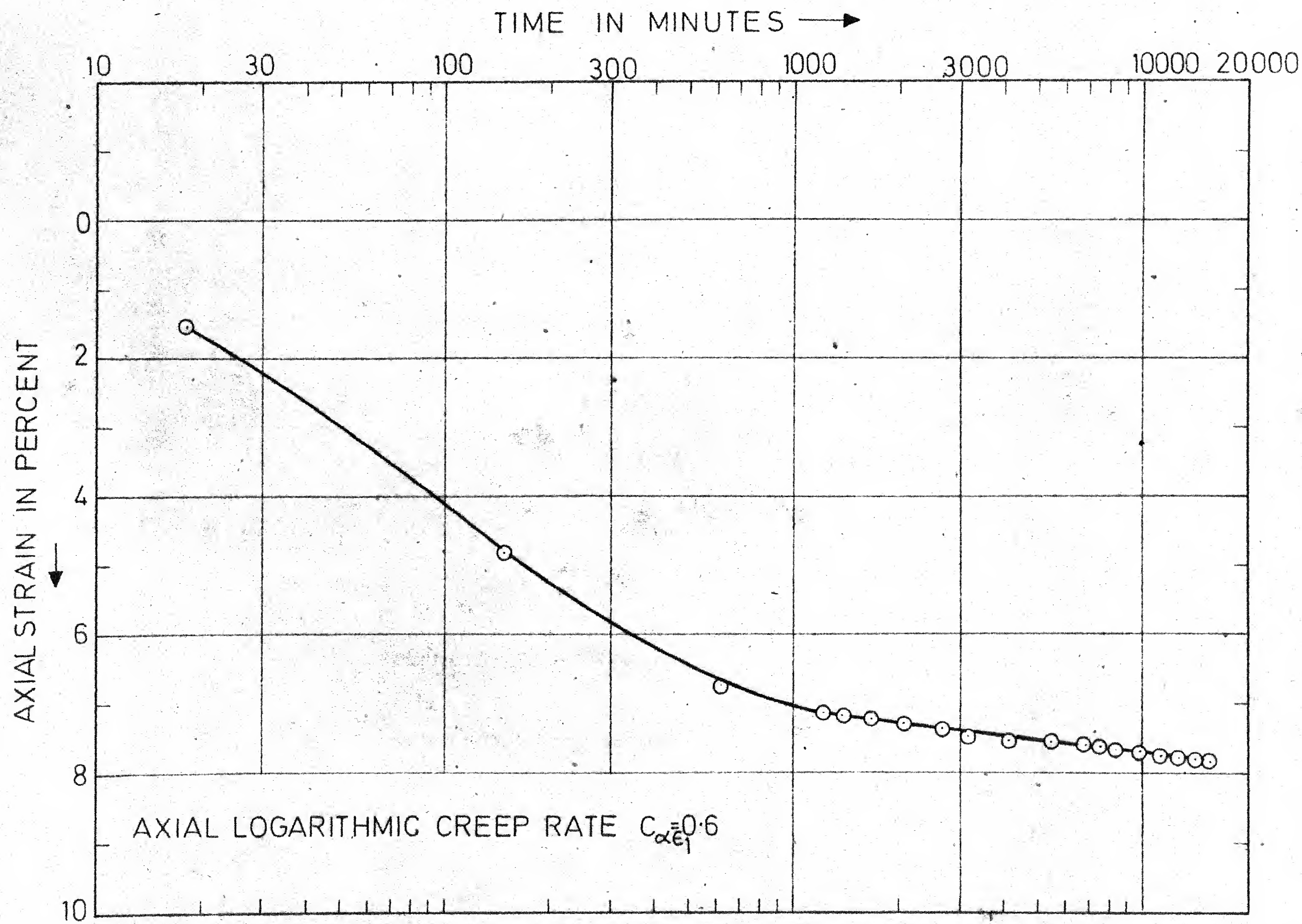


FIG. 5·1 ANISOTROPIC CONSOLIDATION TEST ($q/p' = 0{\cdot}4$) (PRESSURE $P' = 23{\cdot}45$ PSI AND $\sigma_1' = 29{\cdot}7$ PSI)

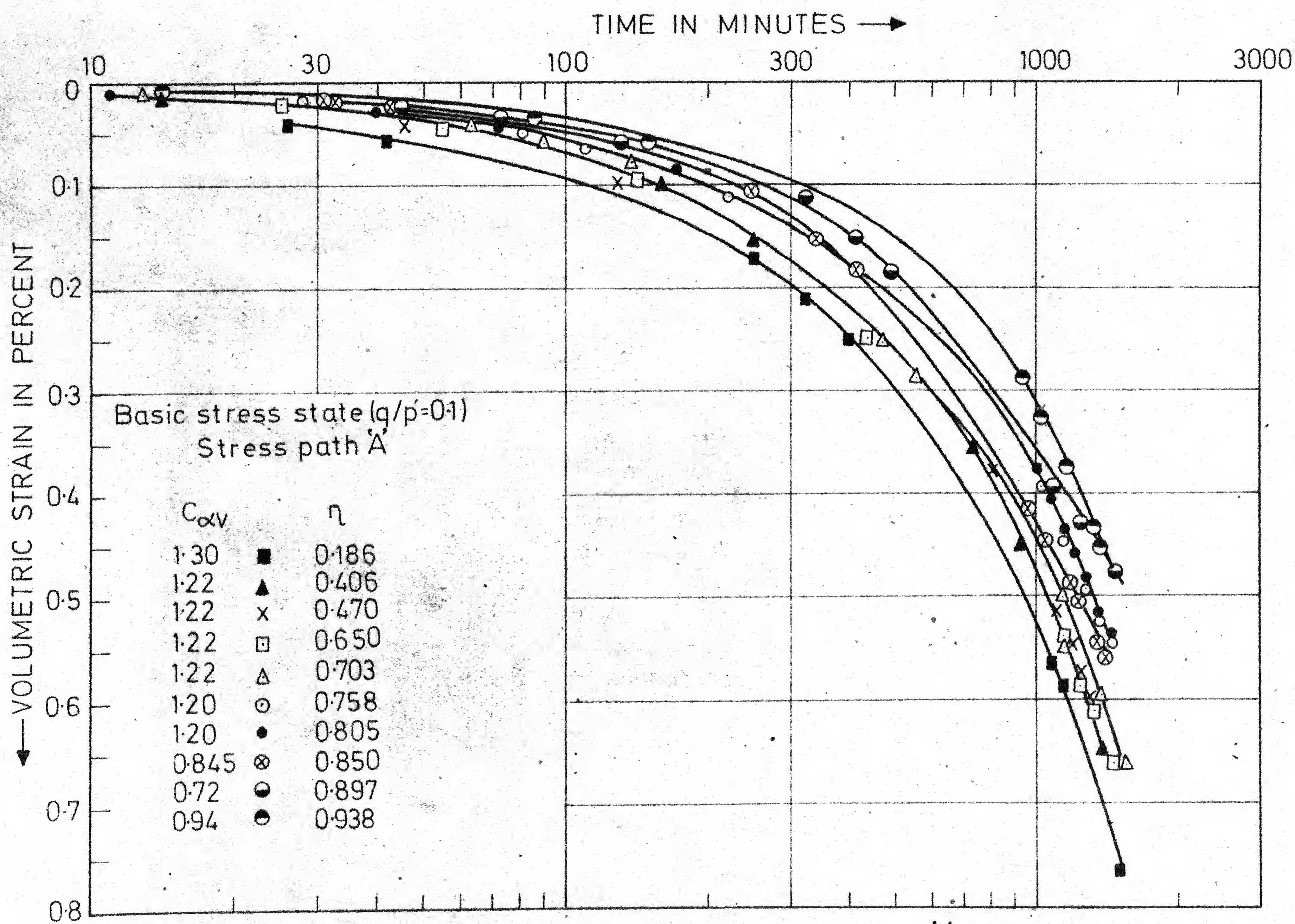


FIG.5·2 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

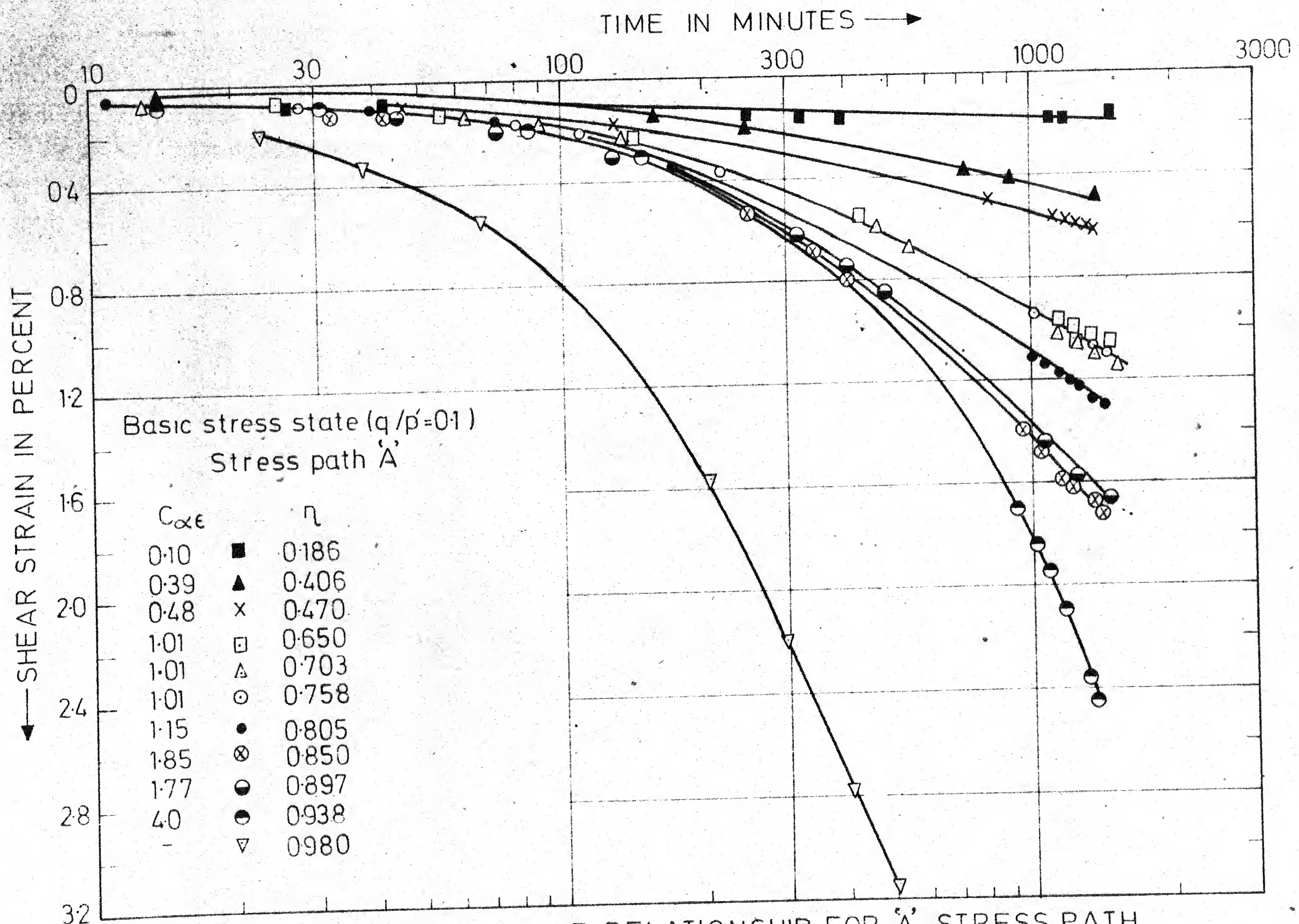


FIG. 5·3 SHEAR STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

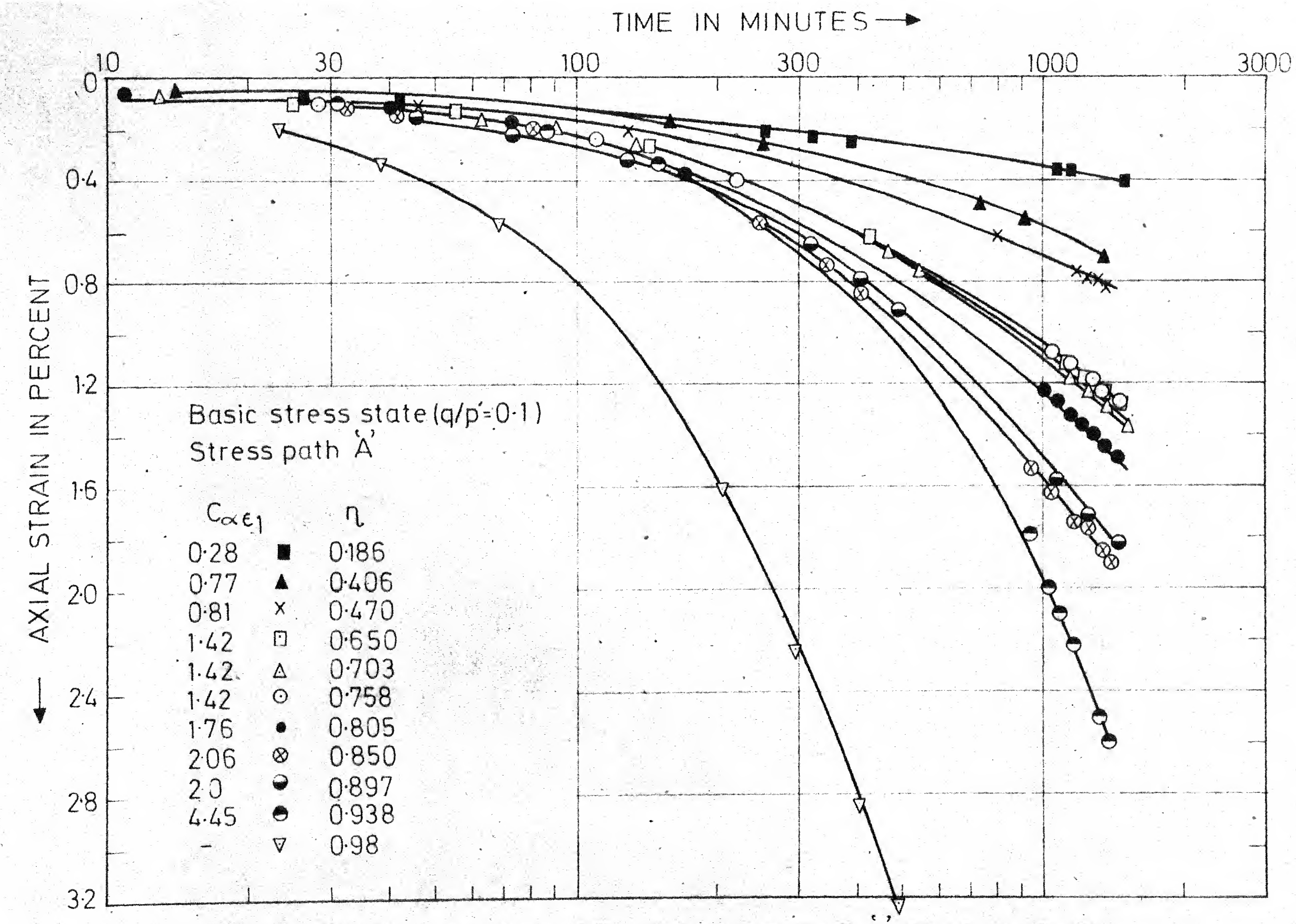


FIG. 5·4 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

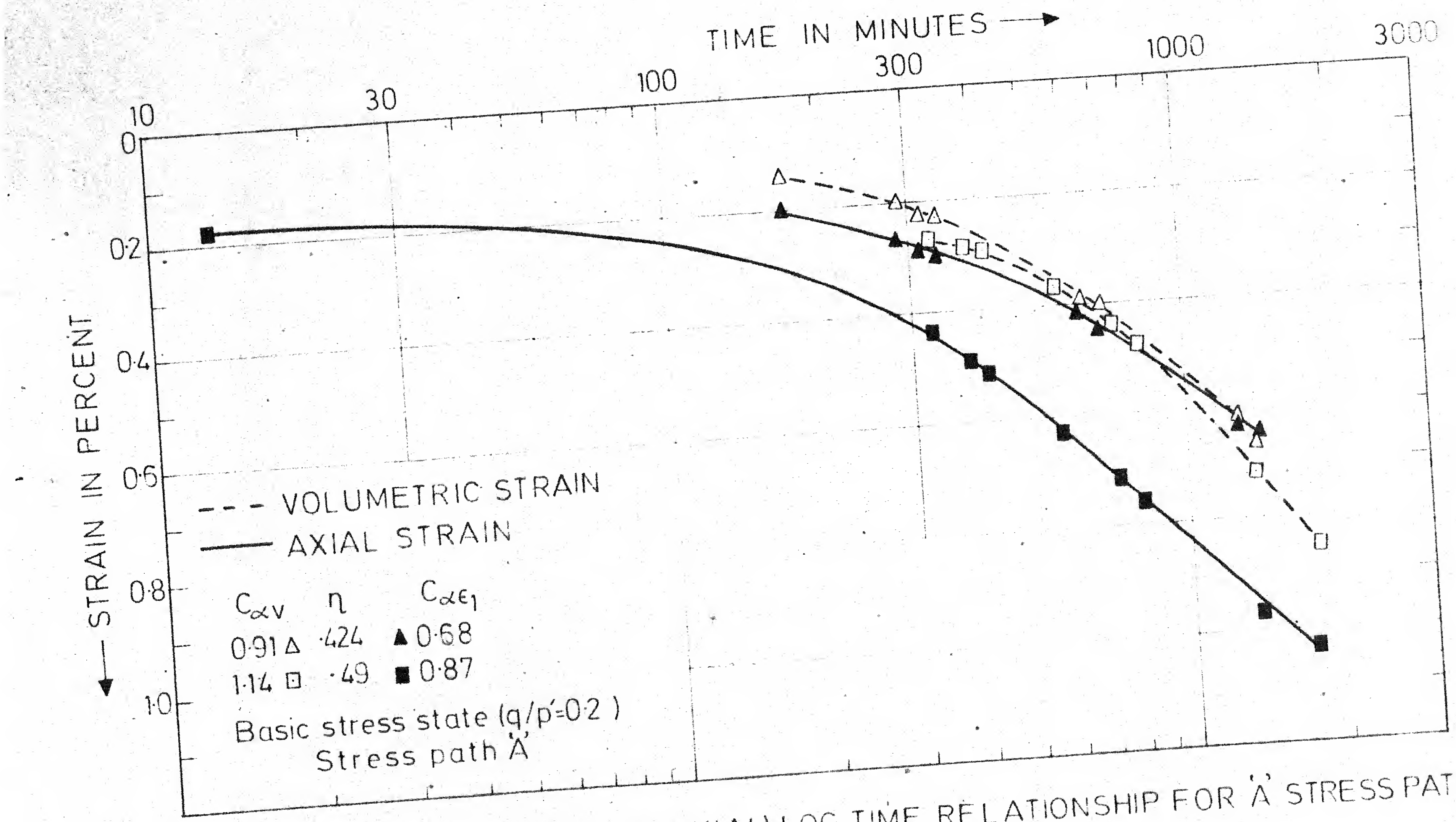


FIG. 5·5 STRAIN(VOLUMETRIC & AXIAL)-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

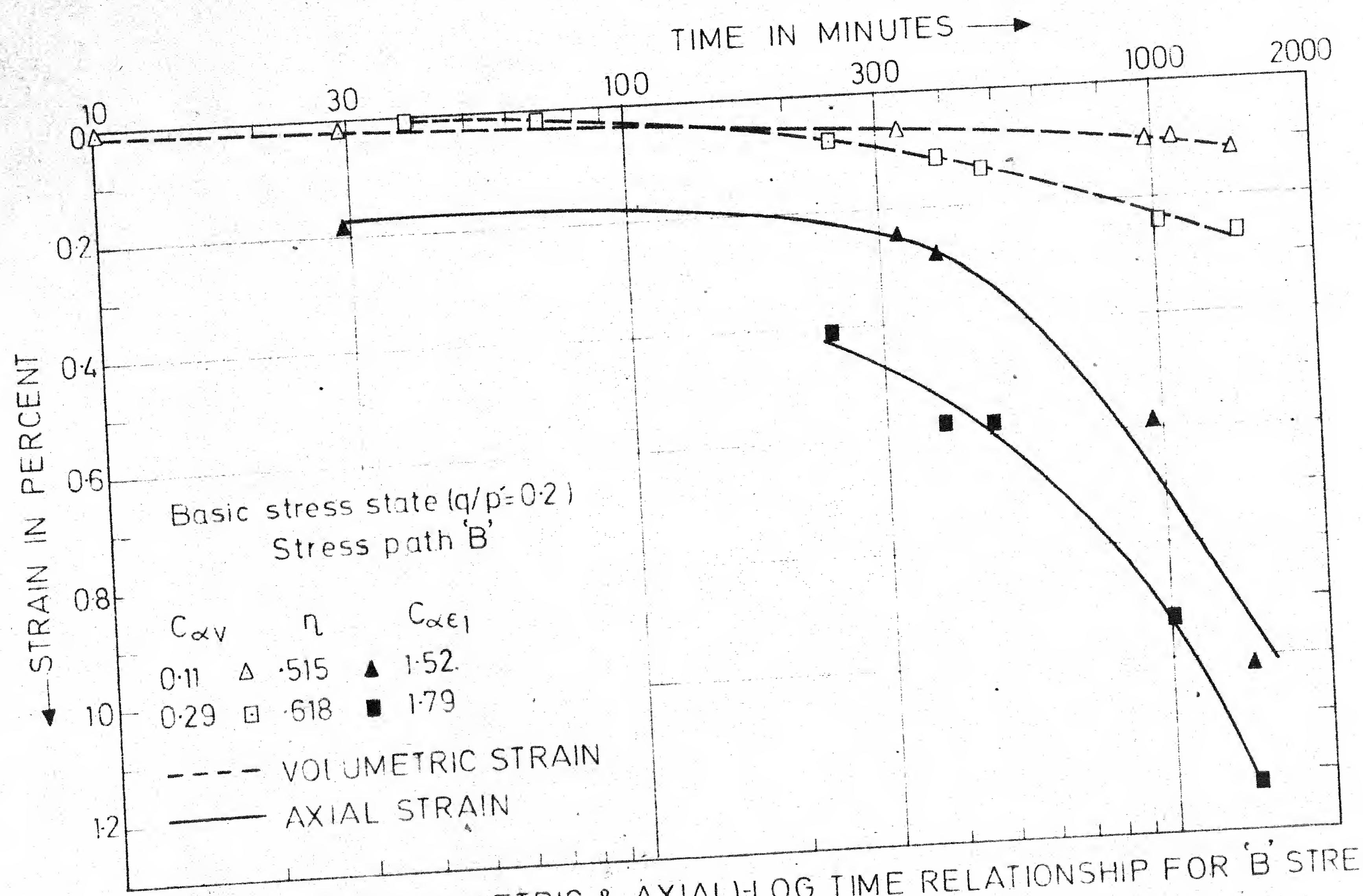


FIG. 5·6 STRAIN (VOLUMETRIC & AXIAL)-LOG TIME RELATIONSHIP FOR 'B' STRESS-PATH

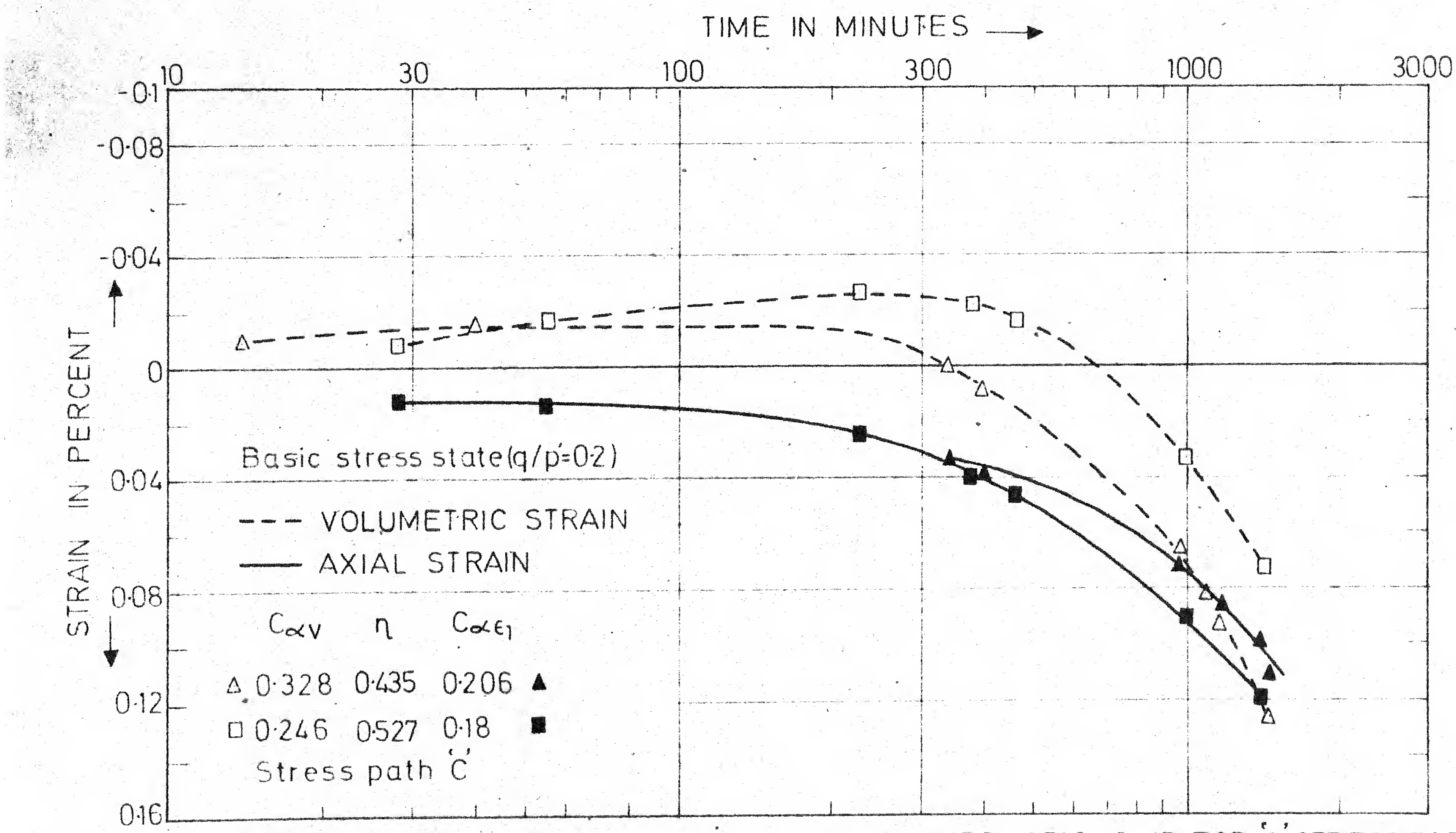


FIG. 5·7 STRAIN (VOLUMETRIC & AXIAL)-LOG TIME RELATIONSHIP FOR 'C' STRESS PATH

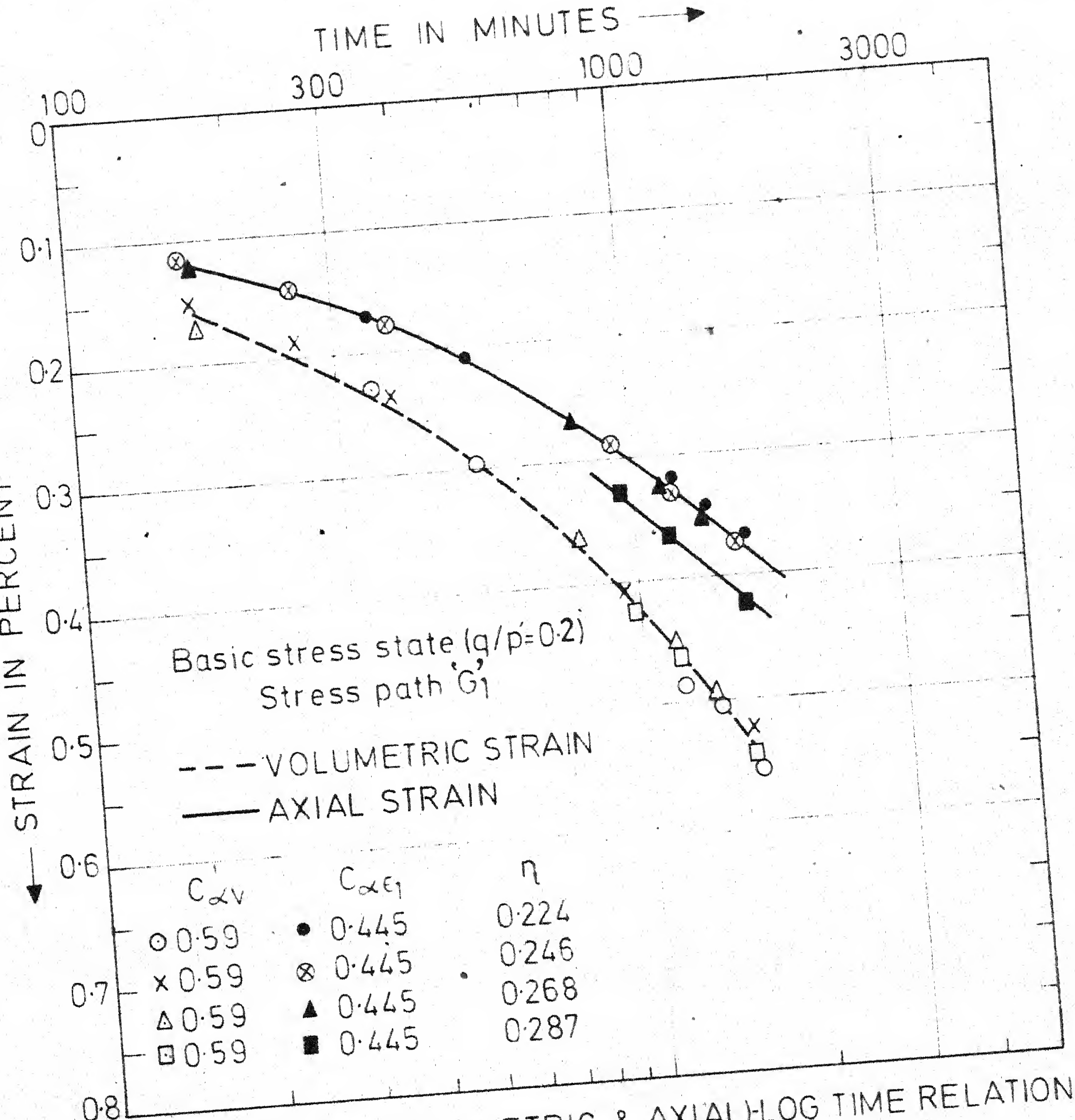


FIG. 5·8 STRAIN (VOLUMETRIC & AXIAL)-LOG TIME RELATIONSHIP FOR 'G1' STRESS PATH

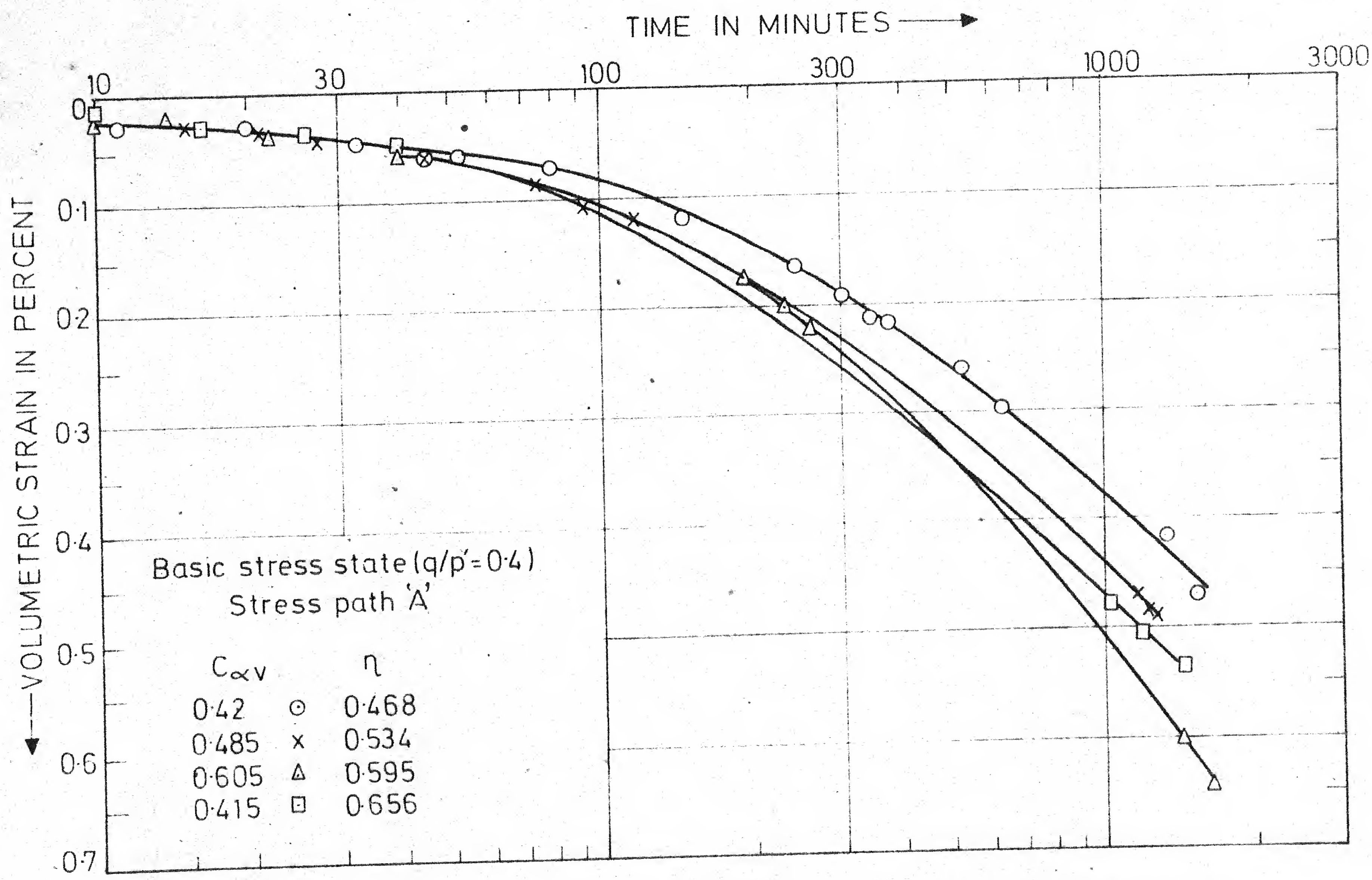


FIG. 5·9 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

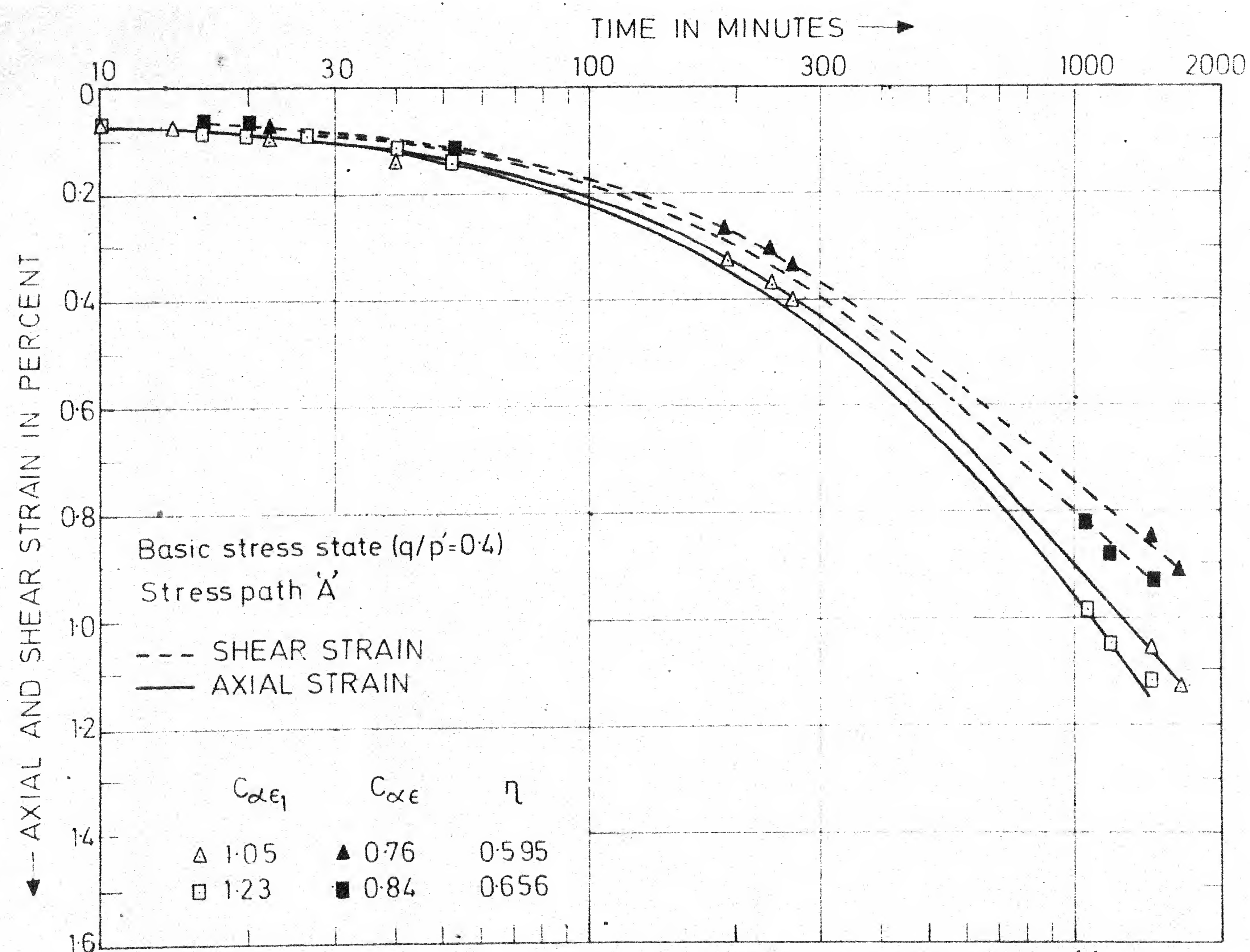


FIG. 5·10 STRAIN (AXIAL & SHEAR)-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

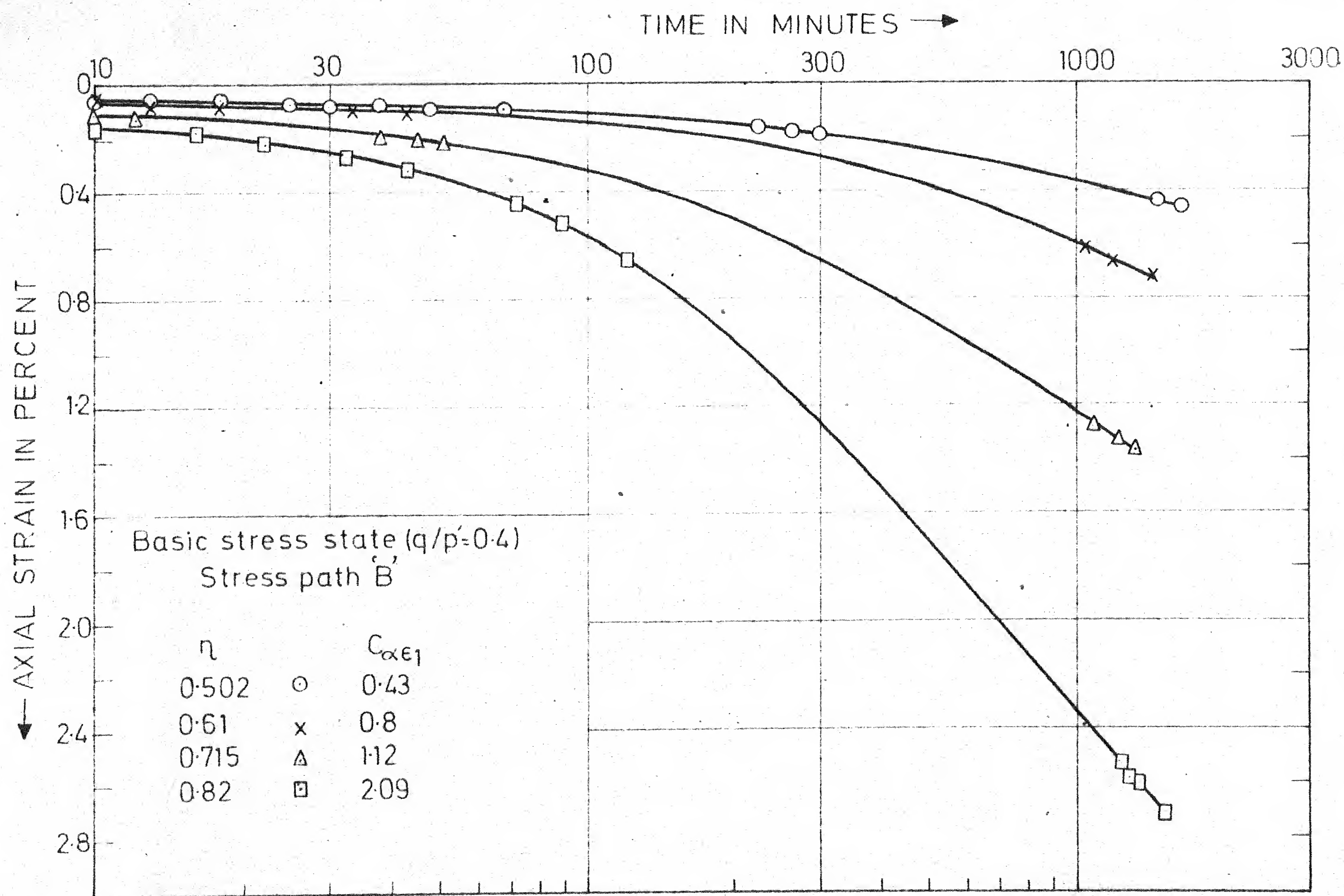
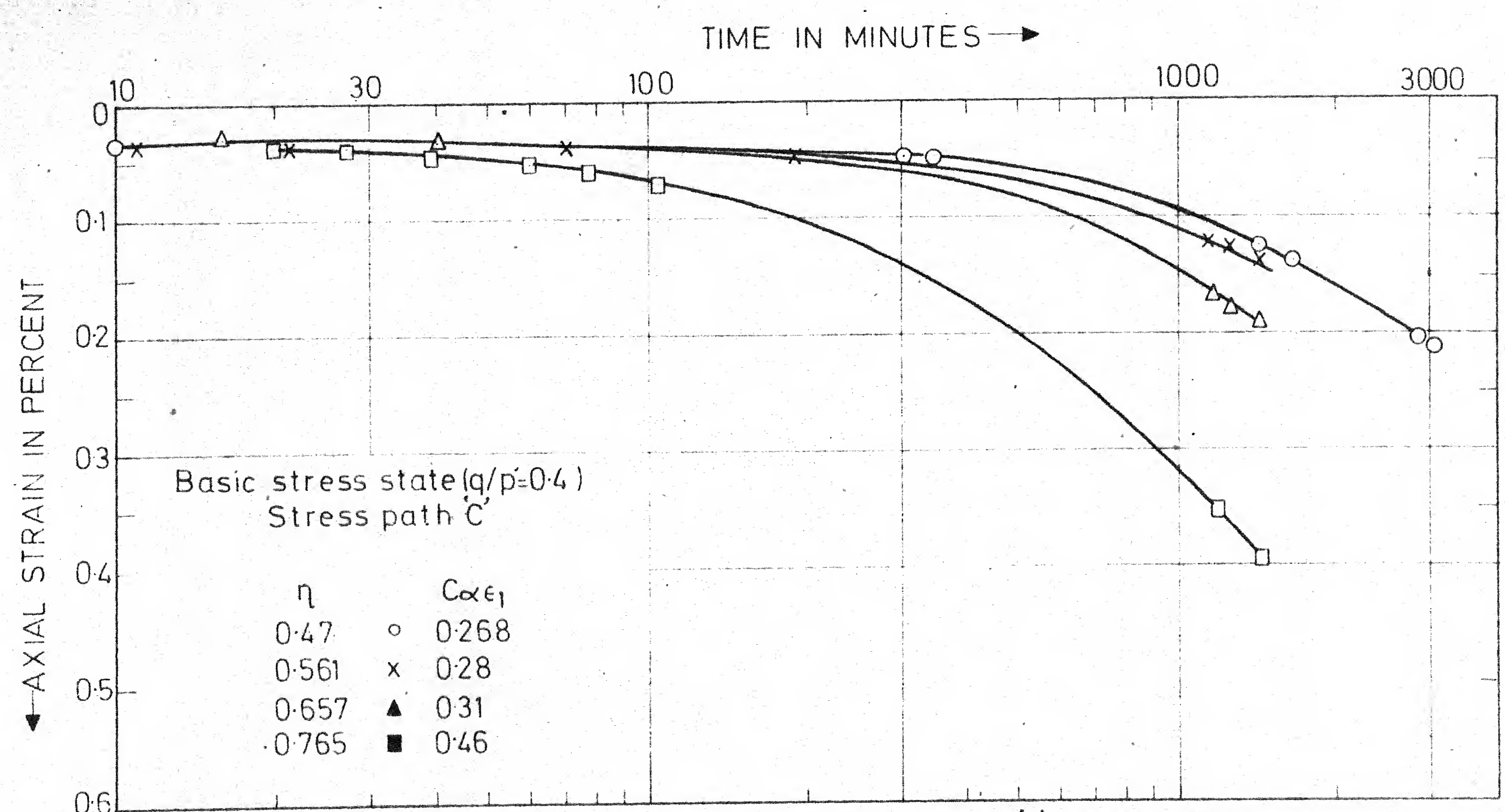
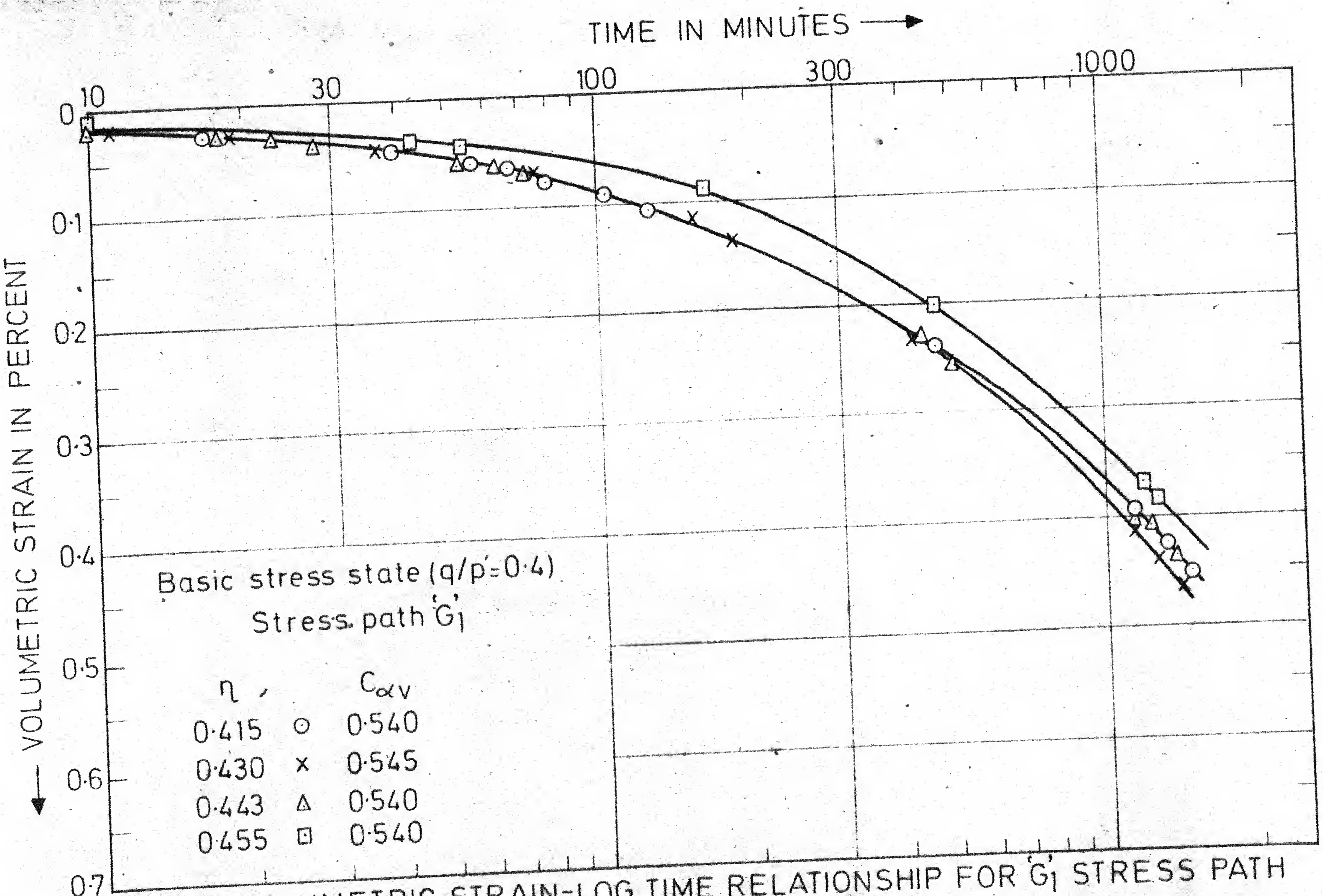


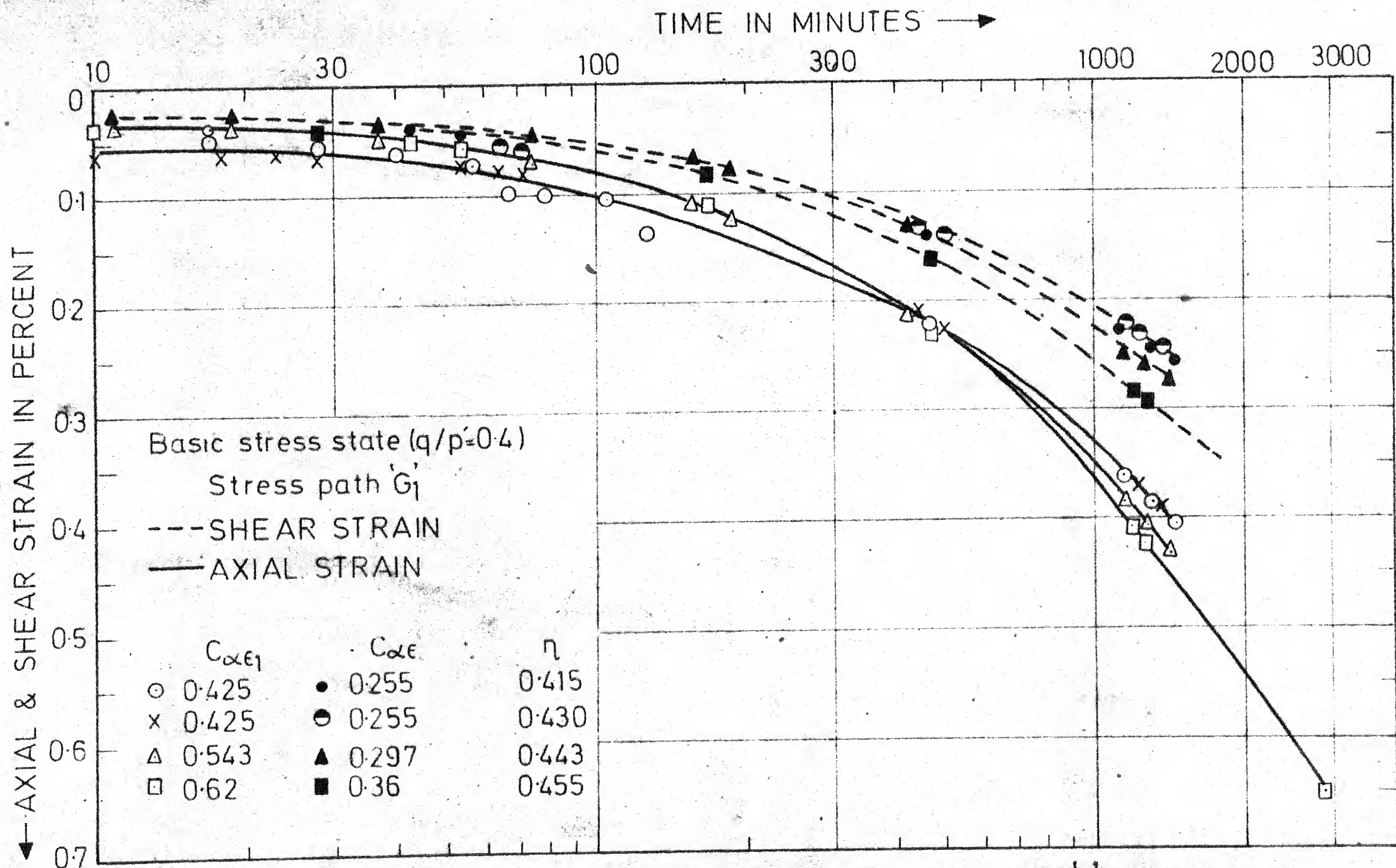
FIG. 5.11 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'B' STRESS PATH



FIG·5·12 AXIAL STRAIN - LOG TIME RELATIONSHIP FOR 'C' STRESS PATH



FIG·5·13 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'G'1 STRESS PATH



FIG·5·14 STRAIN (AXIAL & SHEAR)-LOG TIME RELATIONSHIP FOR 'G1' STRESS PATH

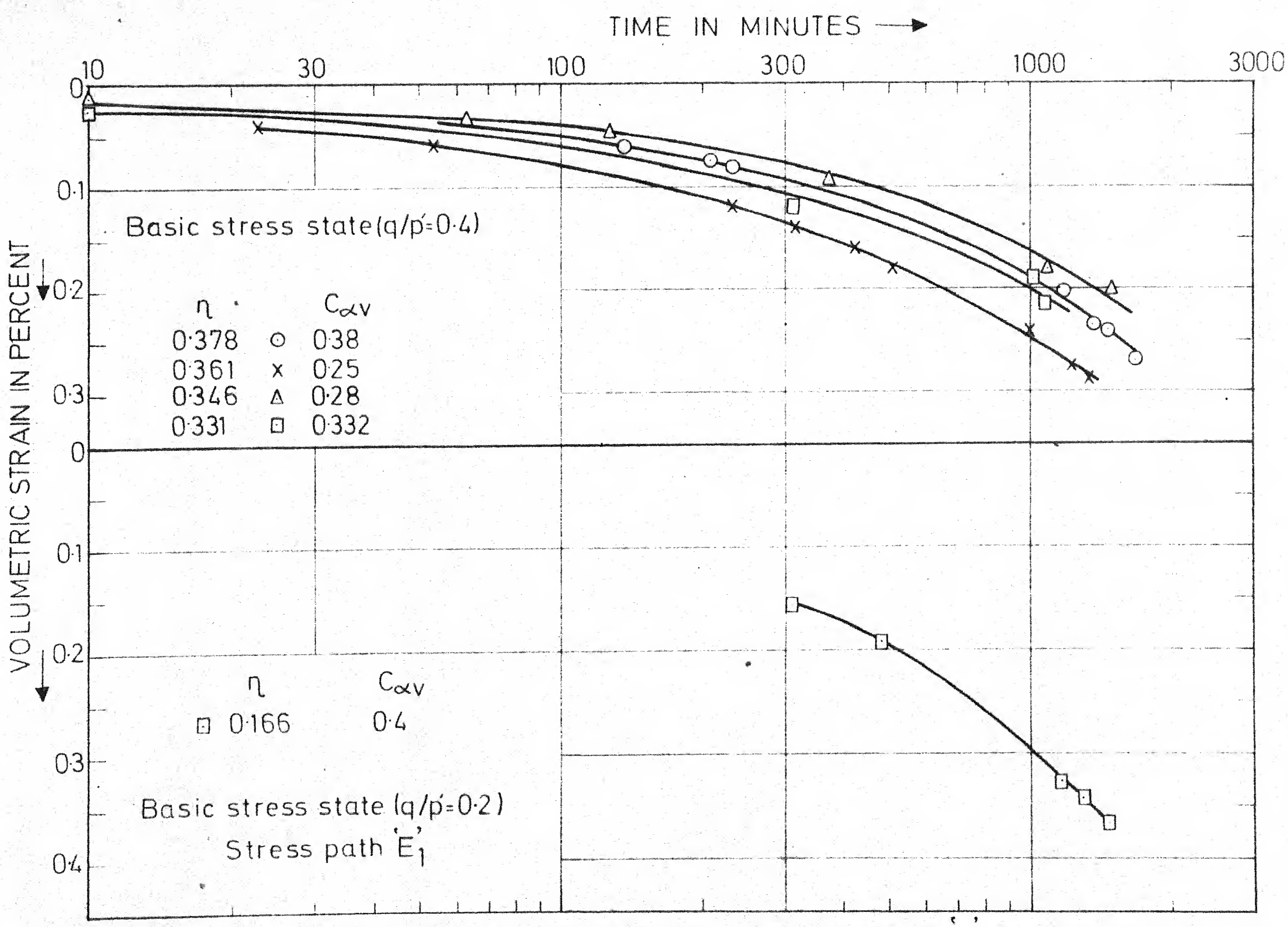


FIG 5.15 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'E1' STRESS PATH

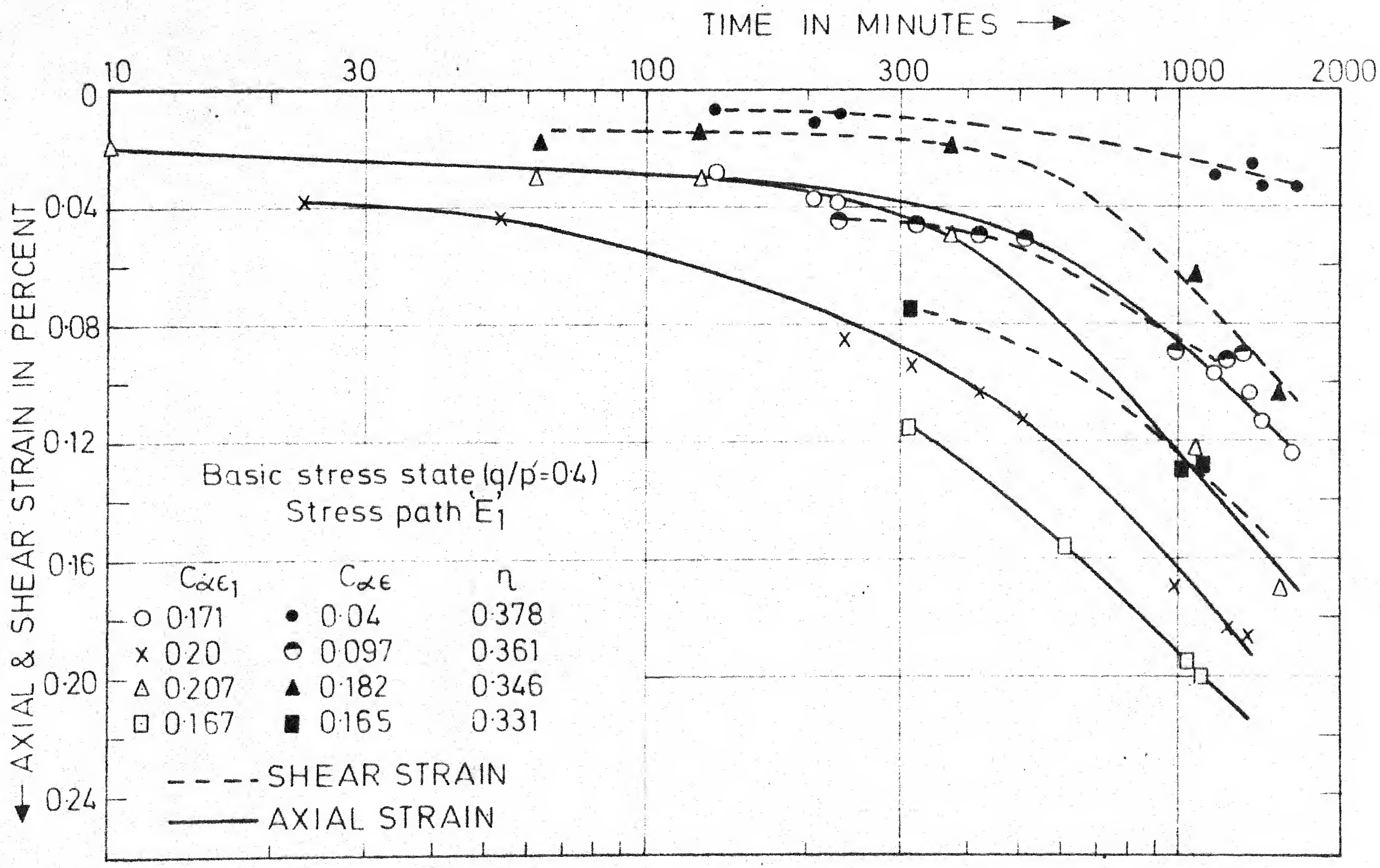
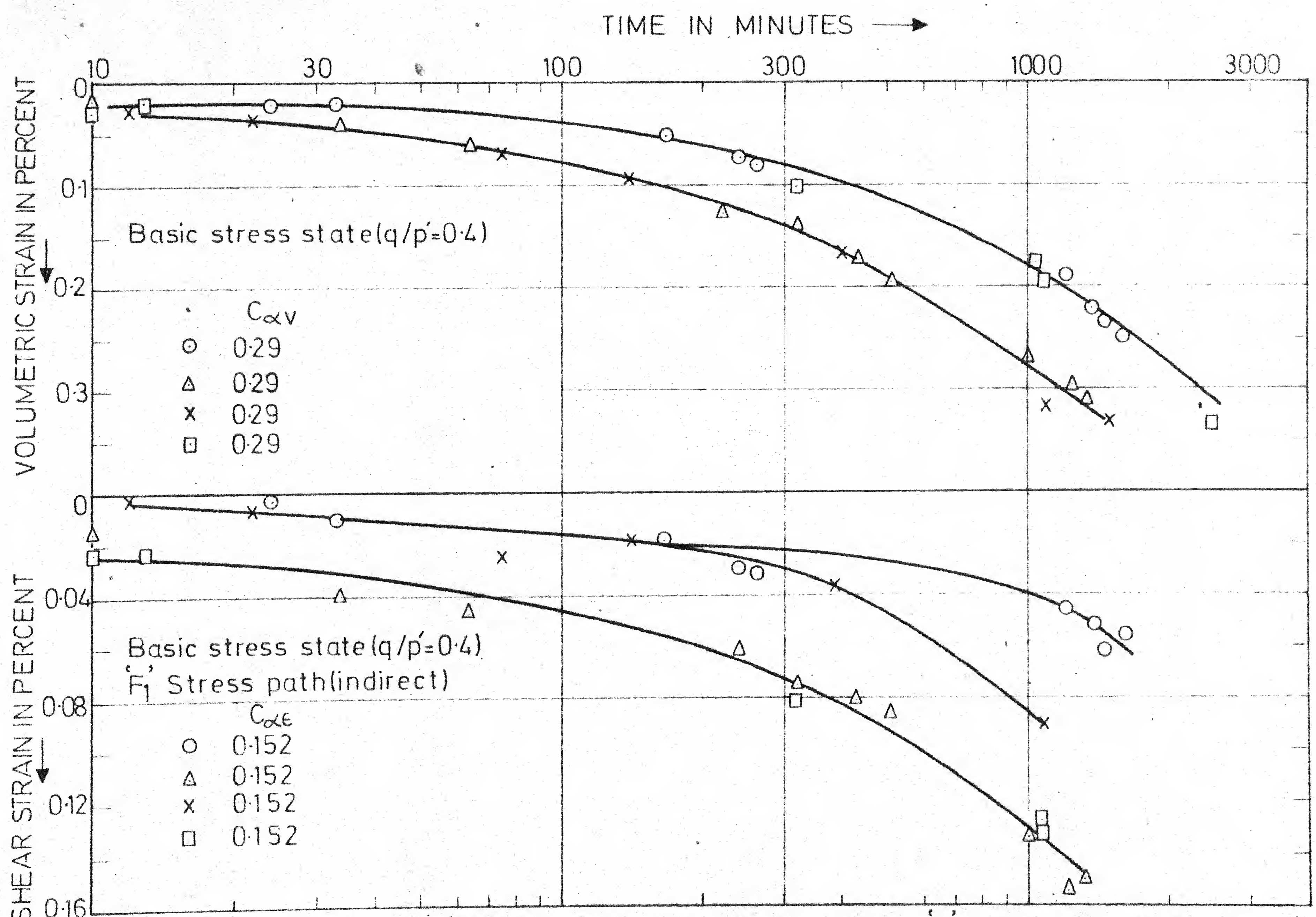


FIG. 5.16 STRAIN (AXIAL & SHEAR)-LOG TIME RELATIONSHIP FOR 'E'1 STRESS PATH



FIG·5·17 VOLUMETRIC STRAIN - LOG TIME RELATIONSHIP FOR 'F1' STRESS PATH (INDIRECT)

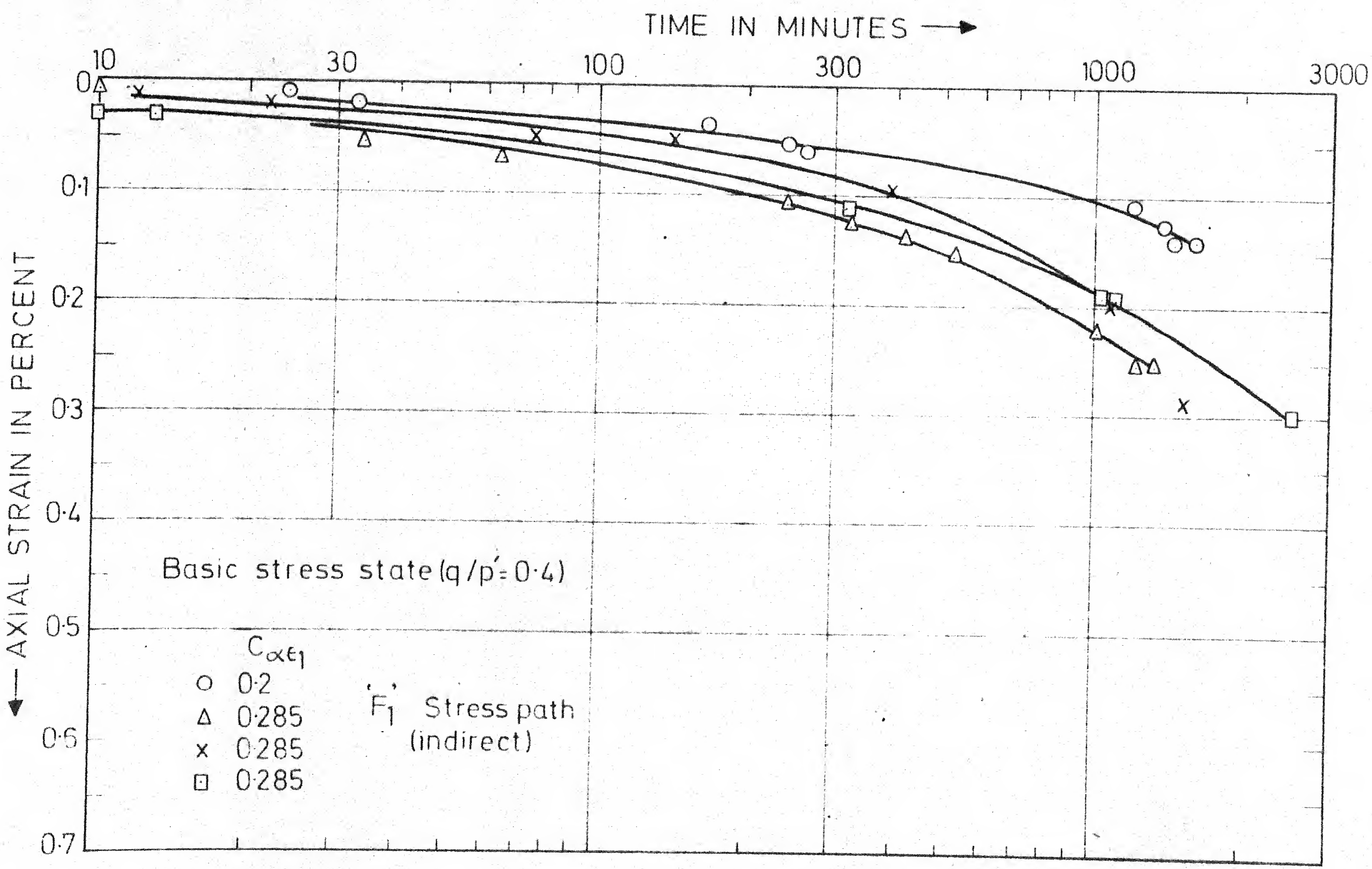


FIG. 5·18 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'F_1' STRESS PATH (INDIRECT)

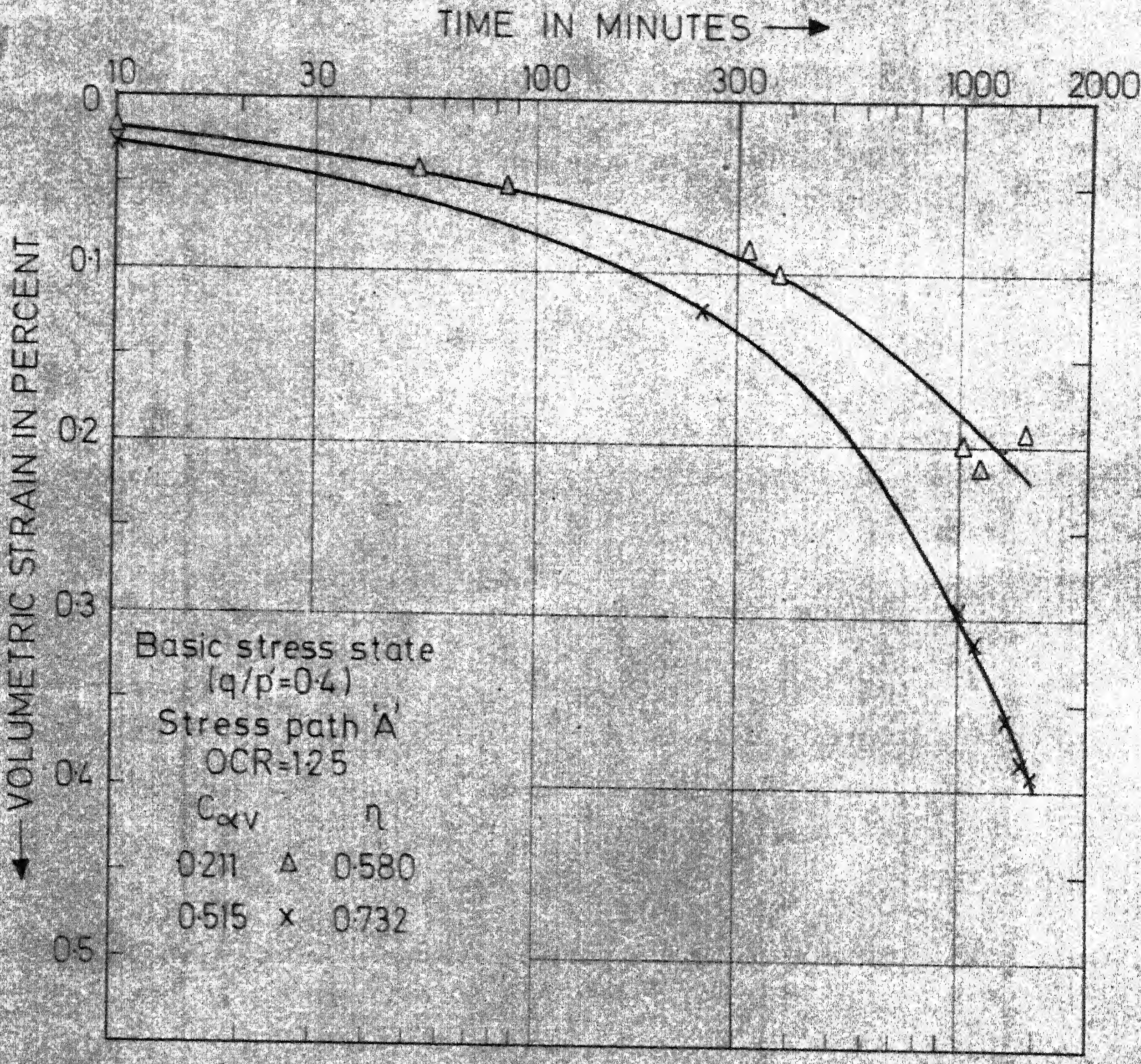


FIG 5·19 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

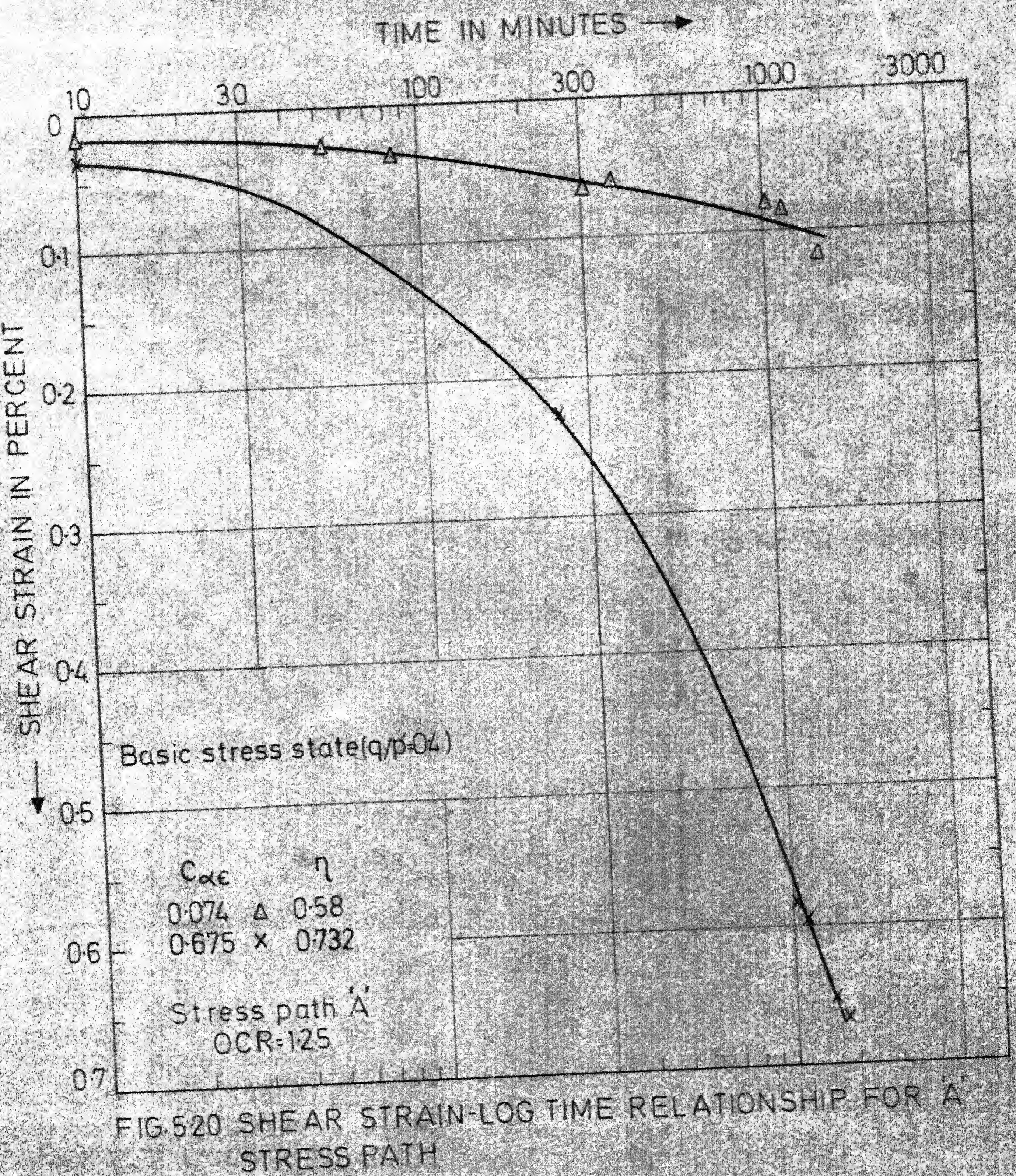


FIG.5·20 SHEAR STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

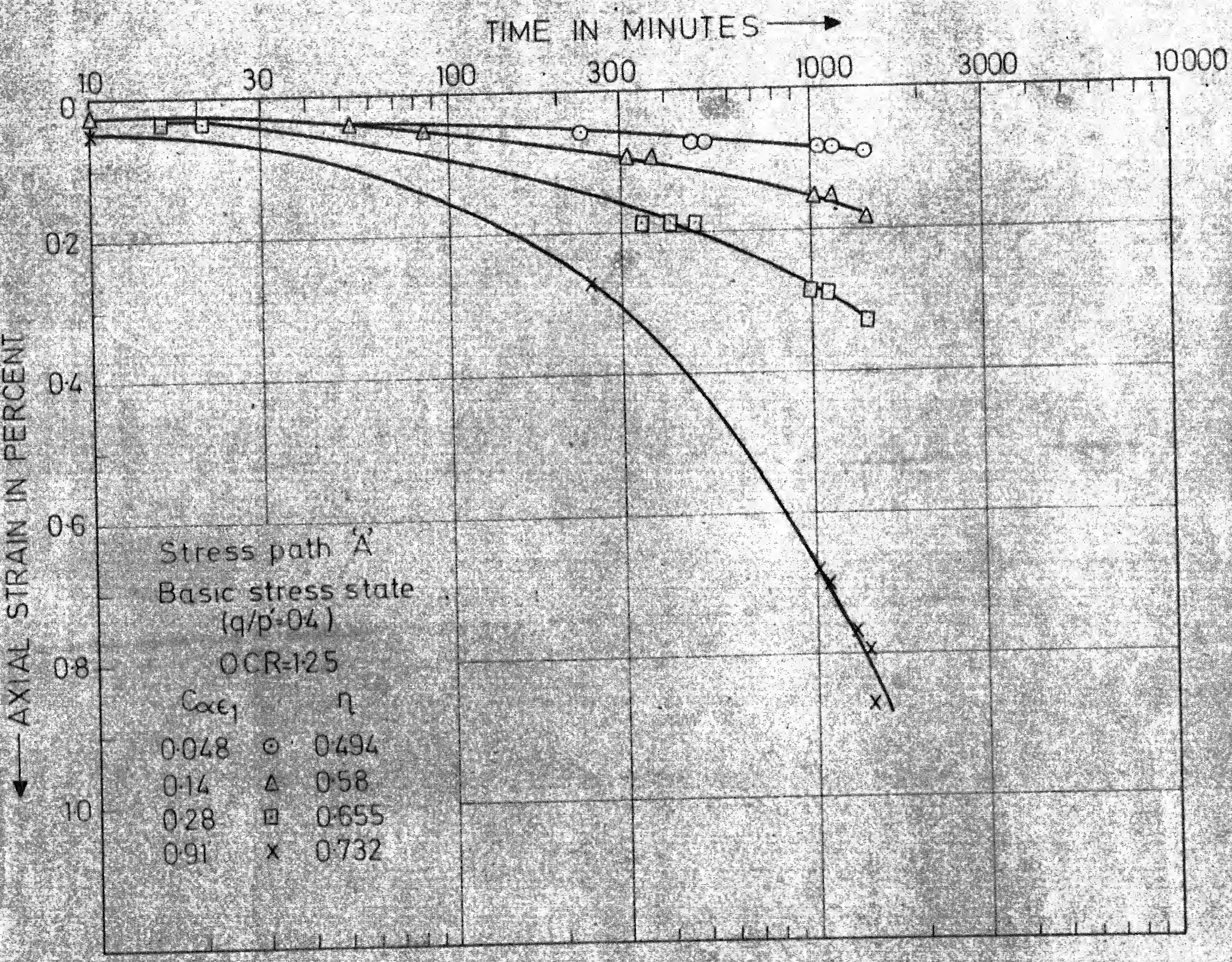
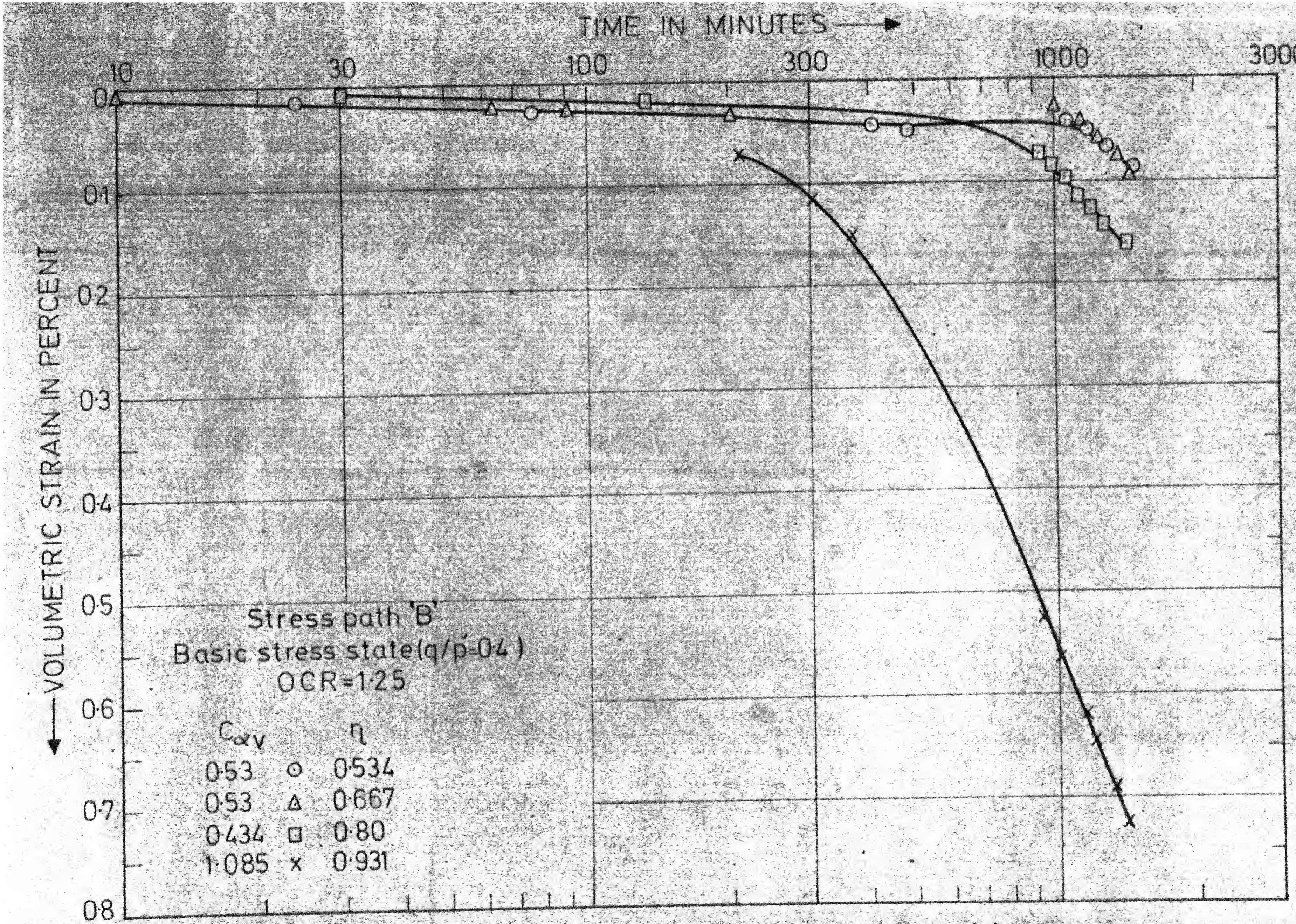


FIG. 5.21 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH



FIG·5·22 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'B' STRESS PATH

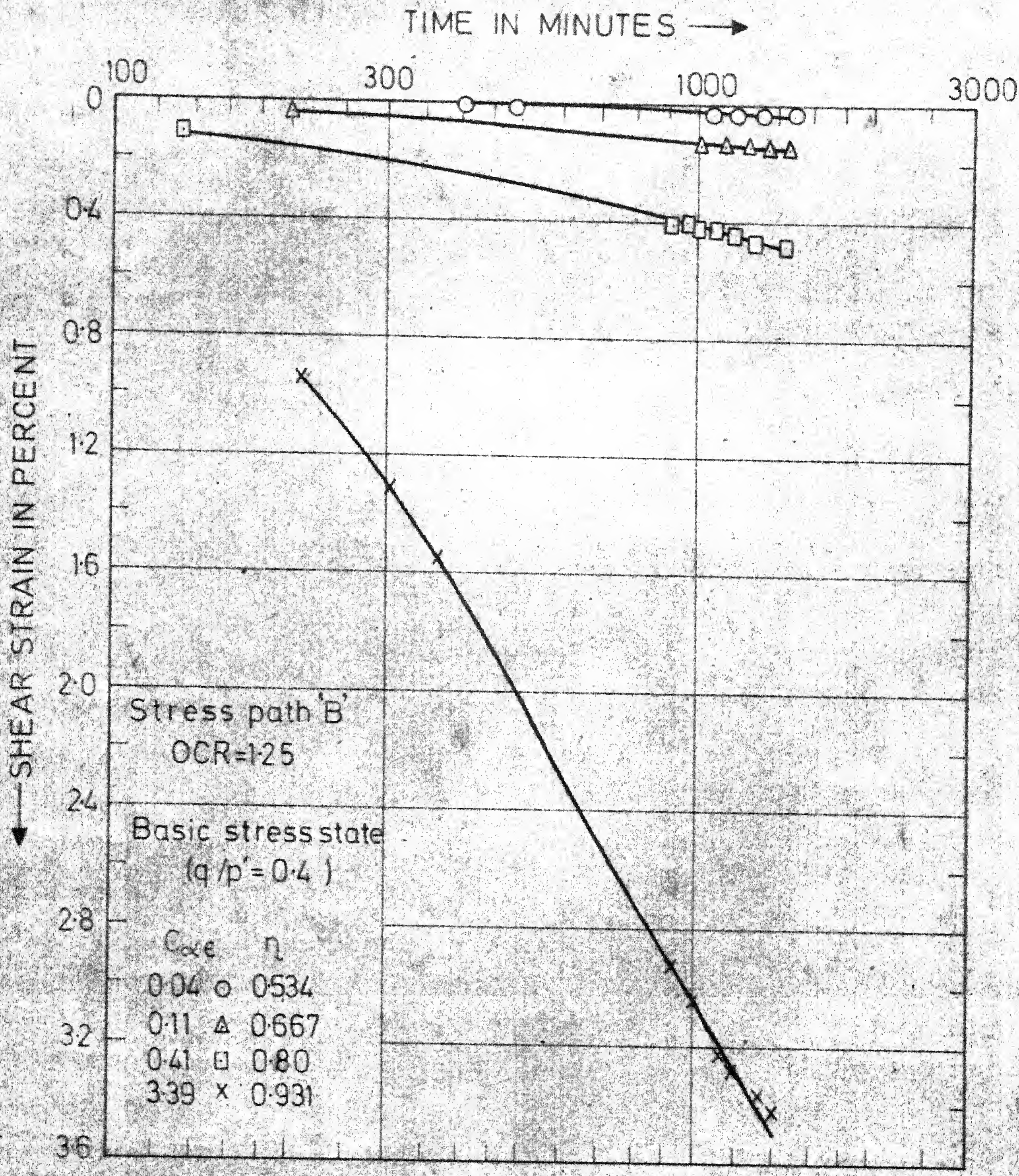


FIG.5.23 SHEAR STRAIN-LOG TIME RELATIONSHIP FOR 'B' STRESS PATH

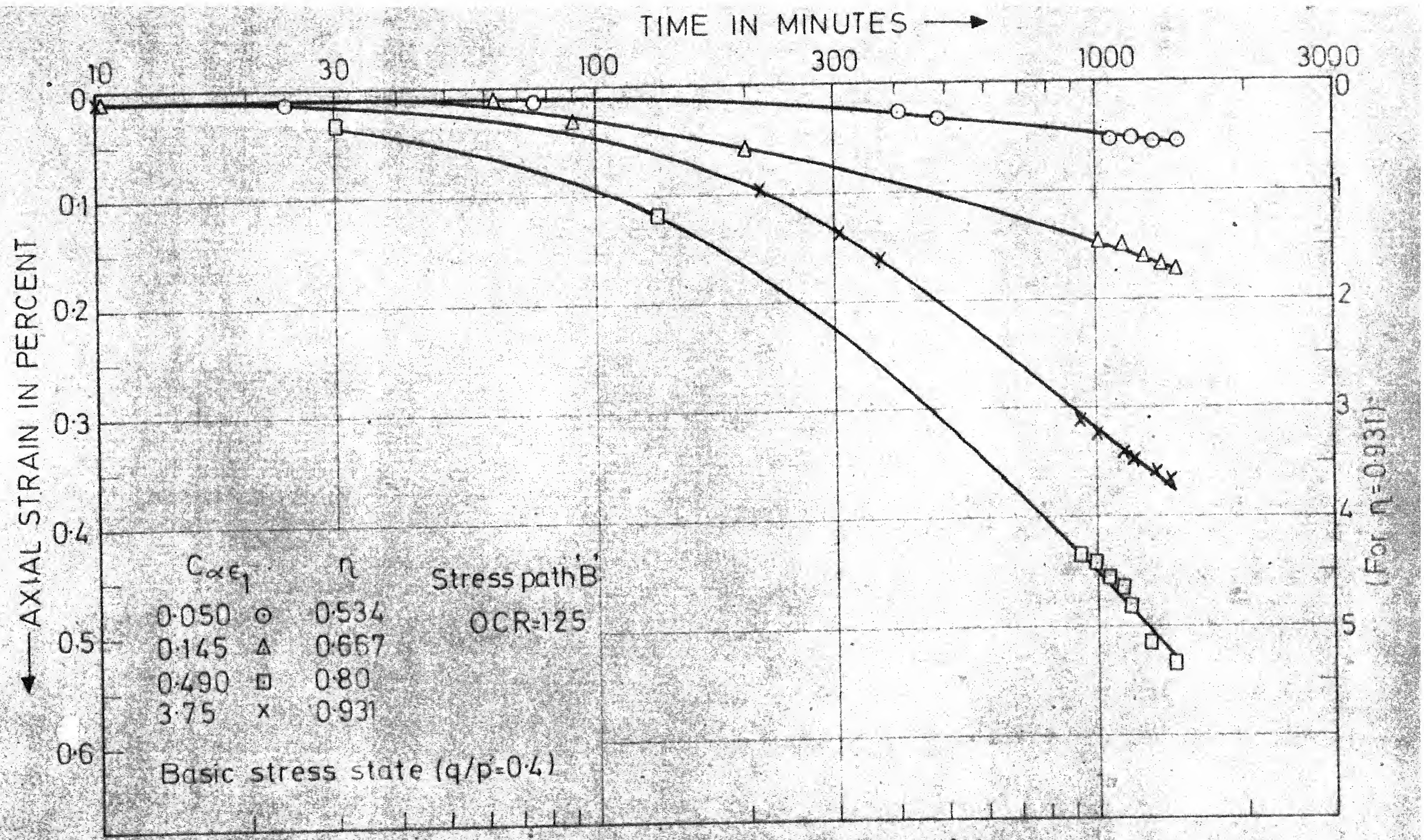


FIG.5·24 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'B' STRESS PATH

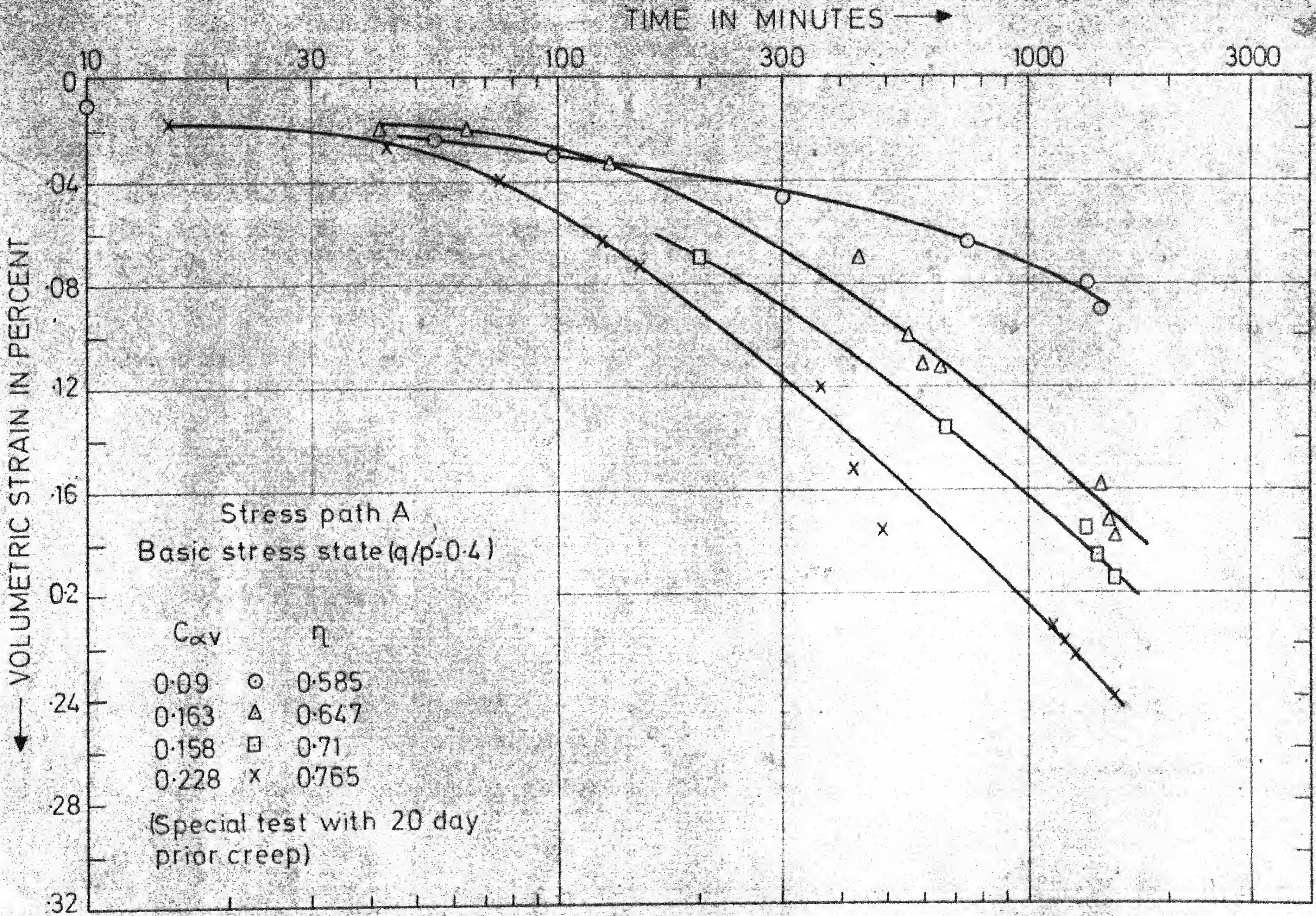


FIG.5·25 VOLUMETRIC STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

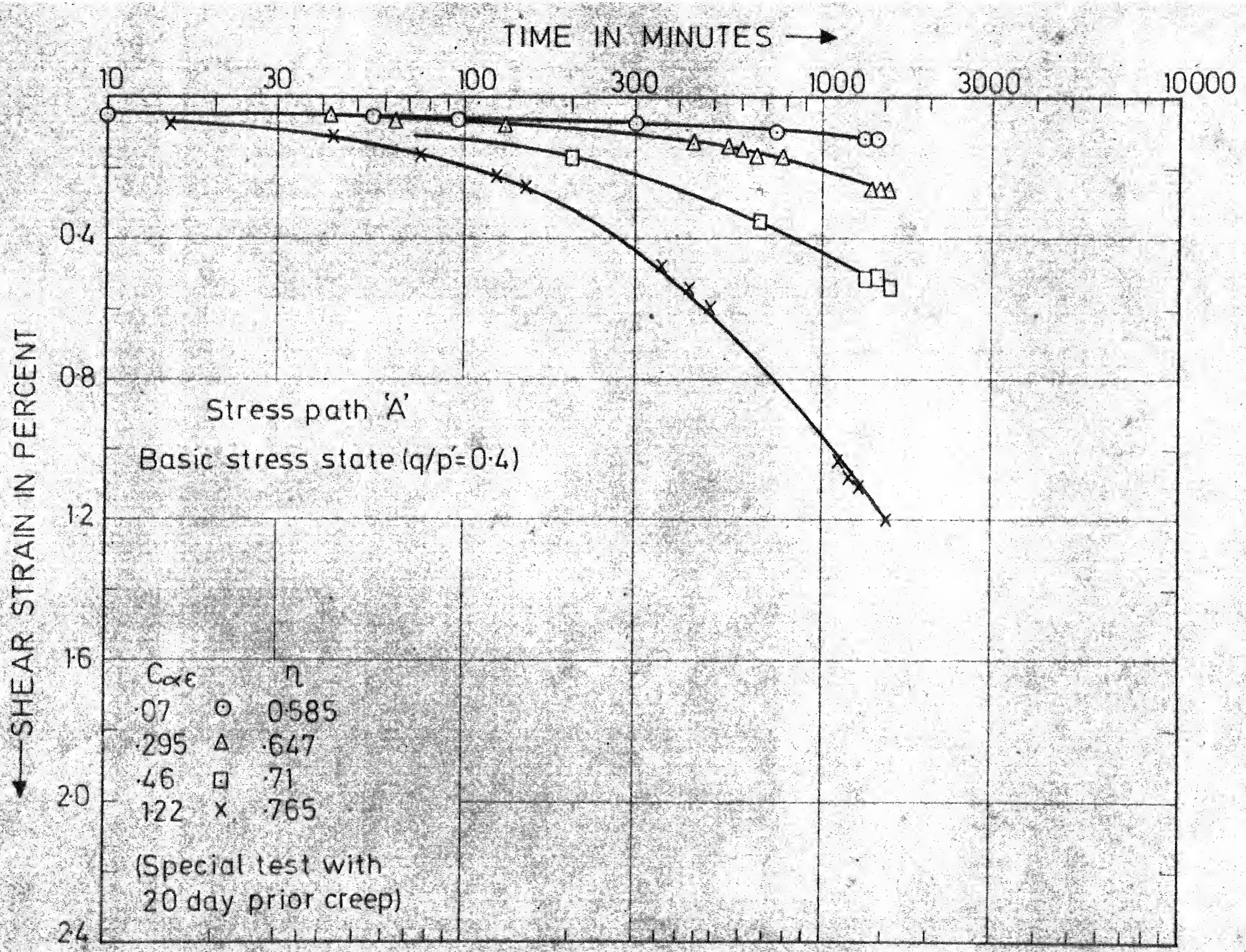


FIG 5·26 SHEAR STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

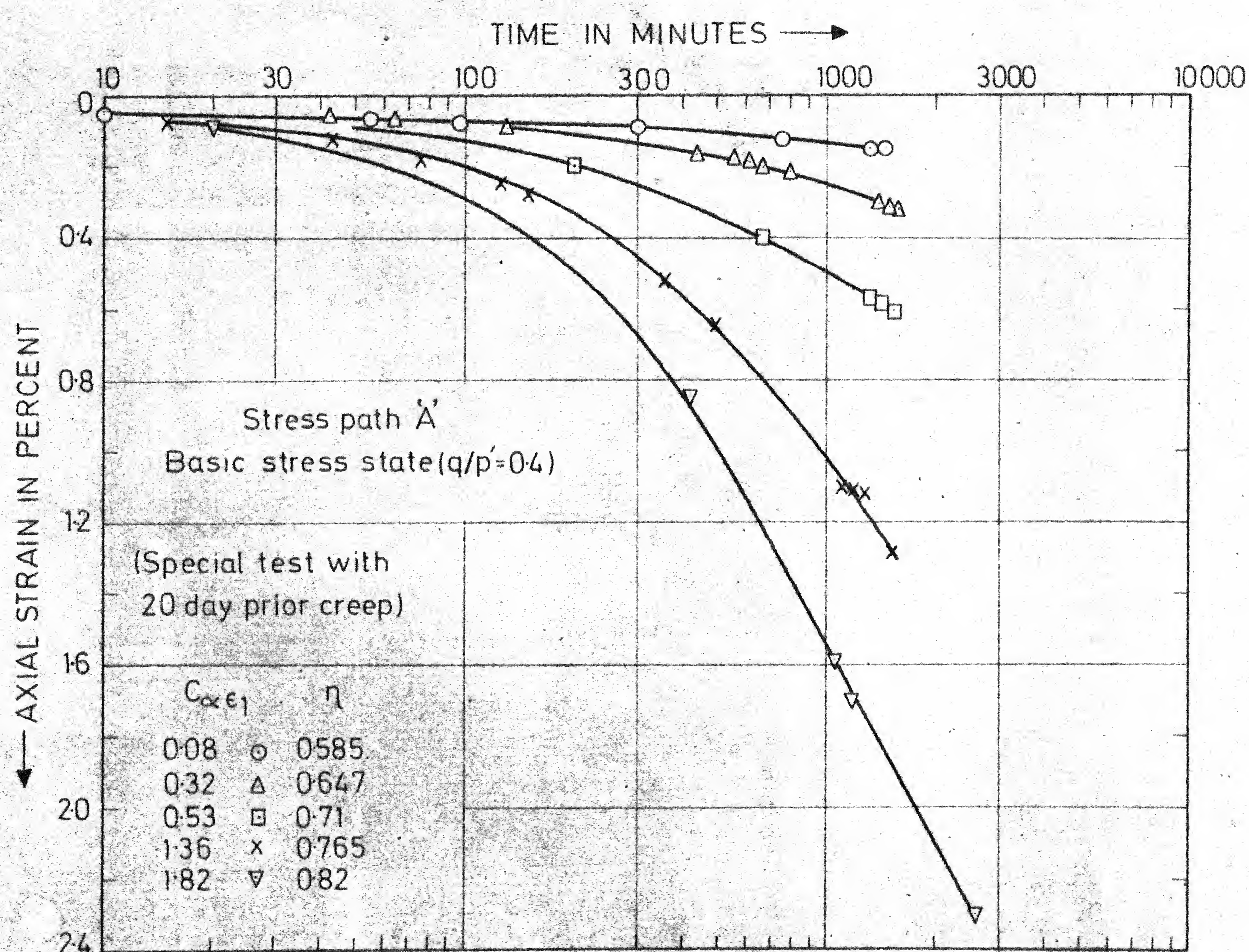


FIG. 5·27 AXIAL STRAIN-LOG TIME RELATIONSHIP FOR 'A' STRESS PATH

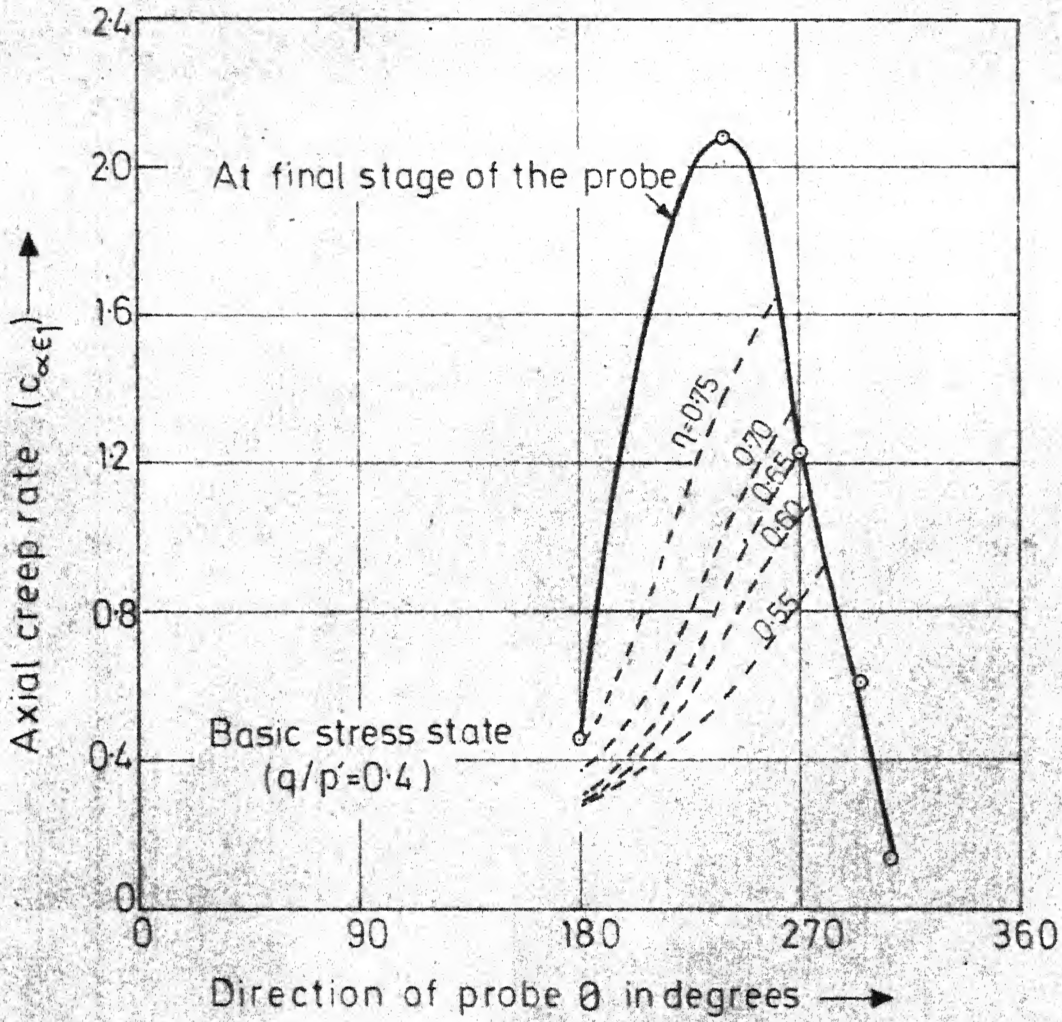


FIG-5·28 VARIATION OF AXIAL CREEP RATE $(C_{\alpha\epsilon_1})$ WITH STRESS PATH (θ) AND STRESS RATIO (η)

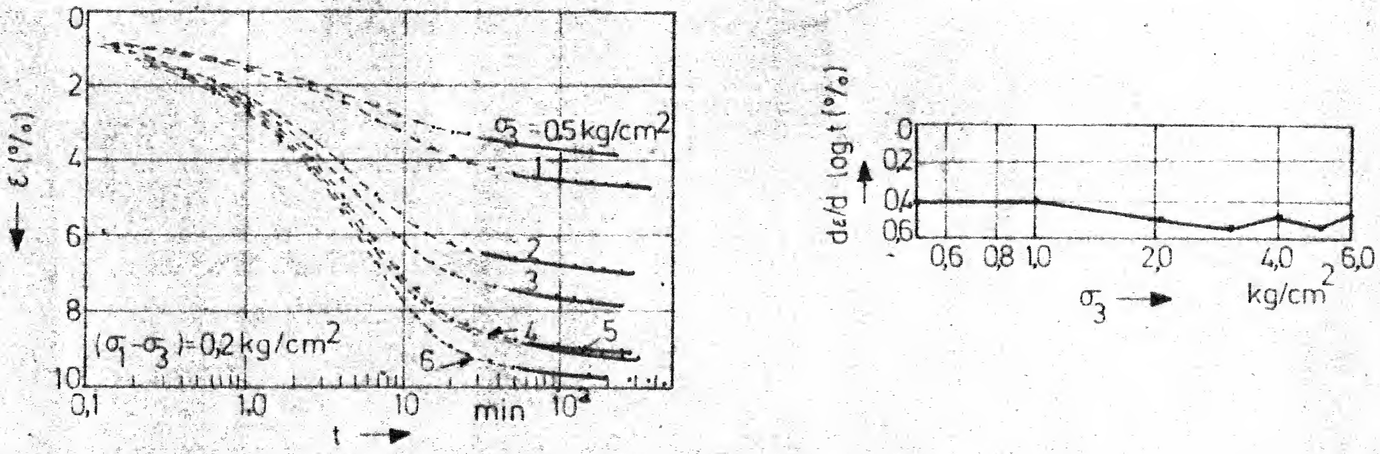


FIG.5.29(a) DRAINED 'E$_1$' STRESS PATH TESTS (AFTER MURAYAMA & SHIBATA 1964)

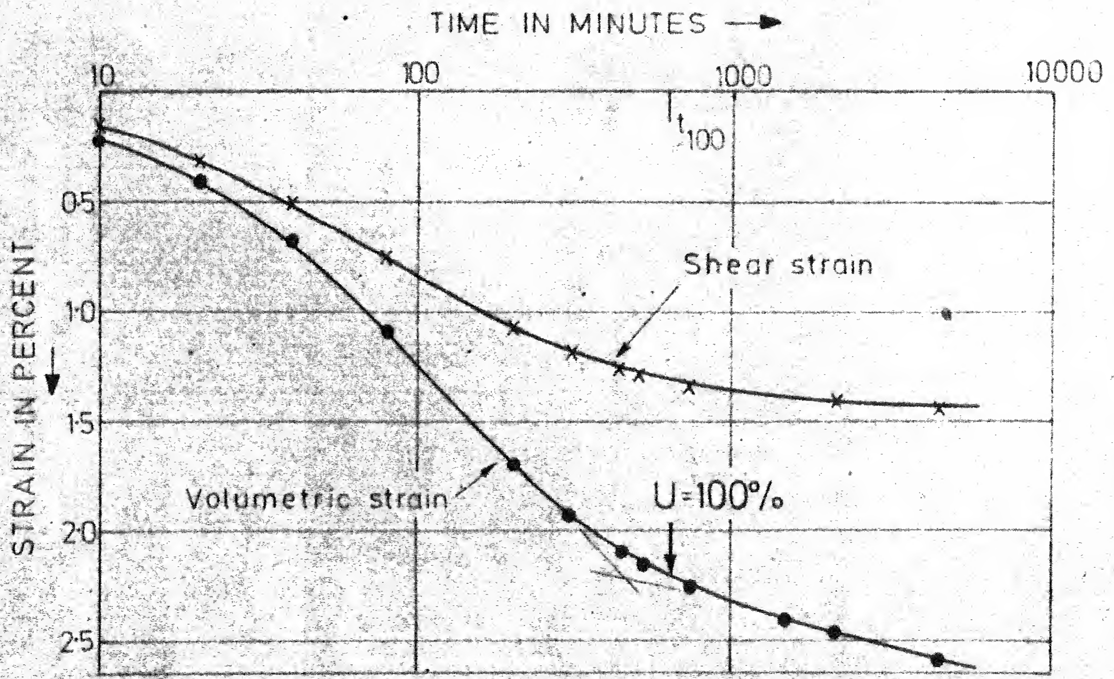
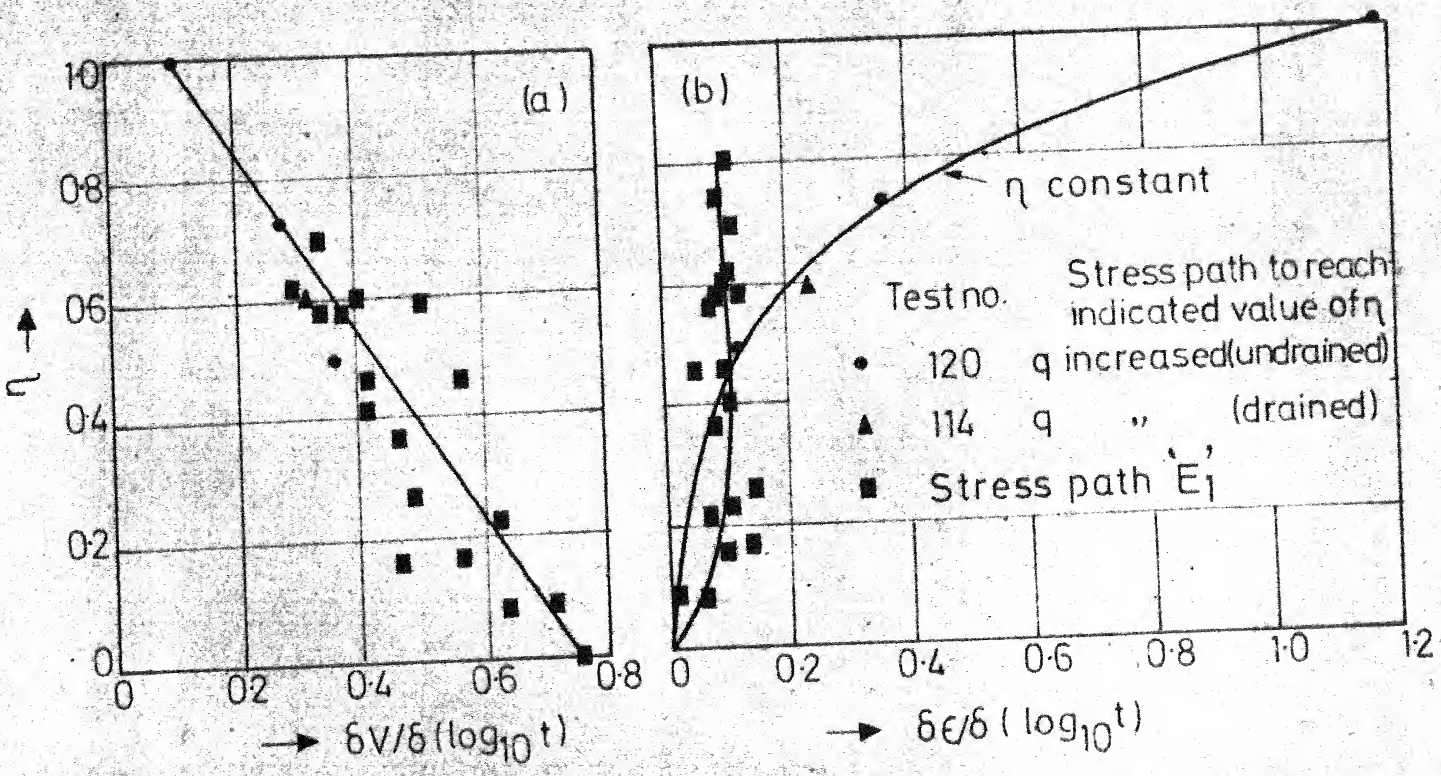


FIG.5.29(b) DRAINED 'E$_1$' STRESS PATH TEST (AFTER BURLAND 1969)



FIG·30 THE RELATIONSHIP BETWEEN STRESS RATIO AND (a) VOLUMETRIC AND (b) SHEAR CREEP RATES (AFTER NEWLAND, 1973)

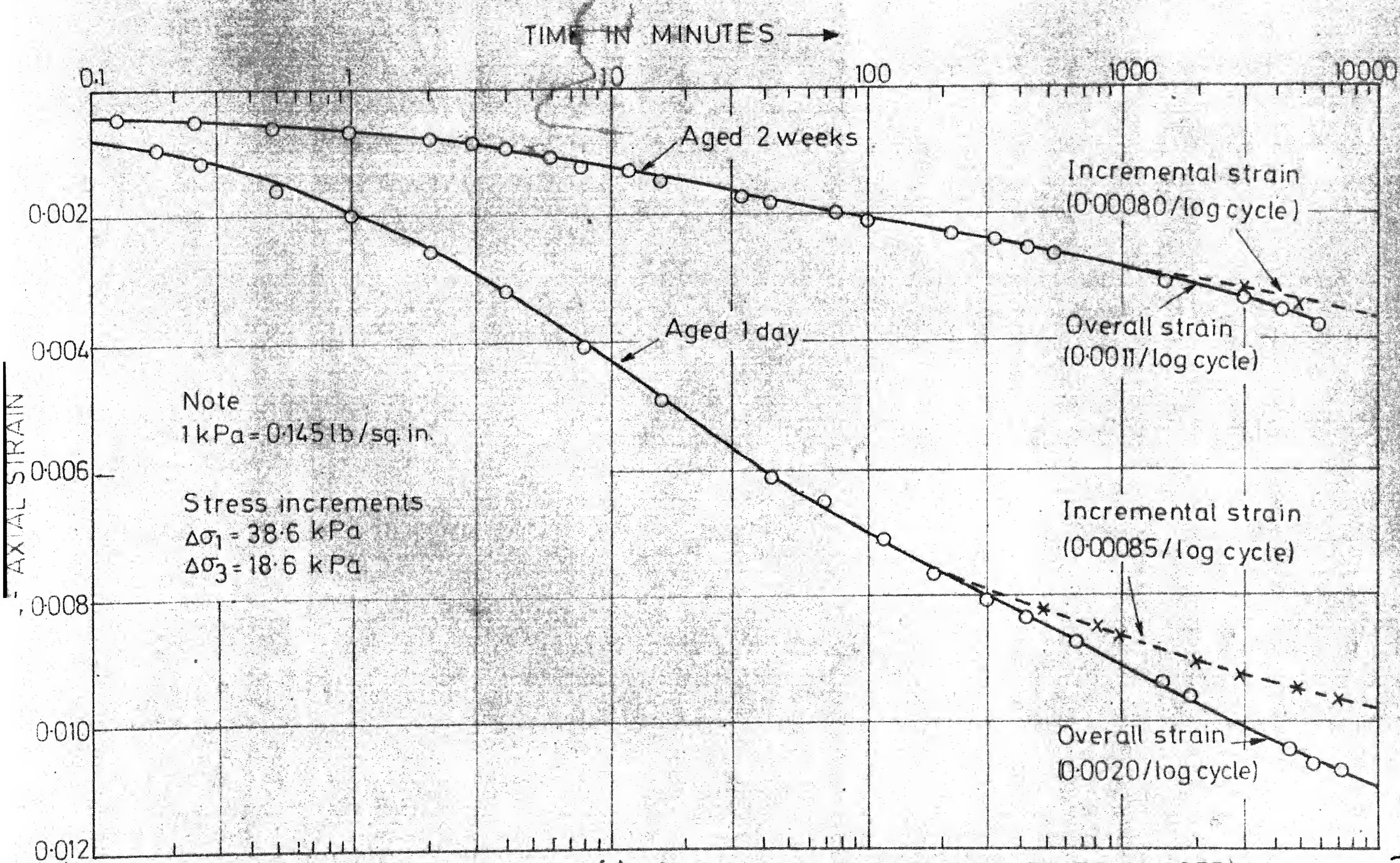


FIG. 5·31 DRAINED CREEP TEST FOR $\acute{G}_1$ STRESS PATH (AFTER POULOS ET. AL. 1975)

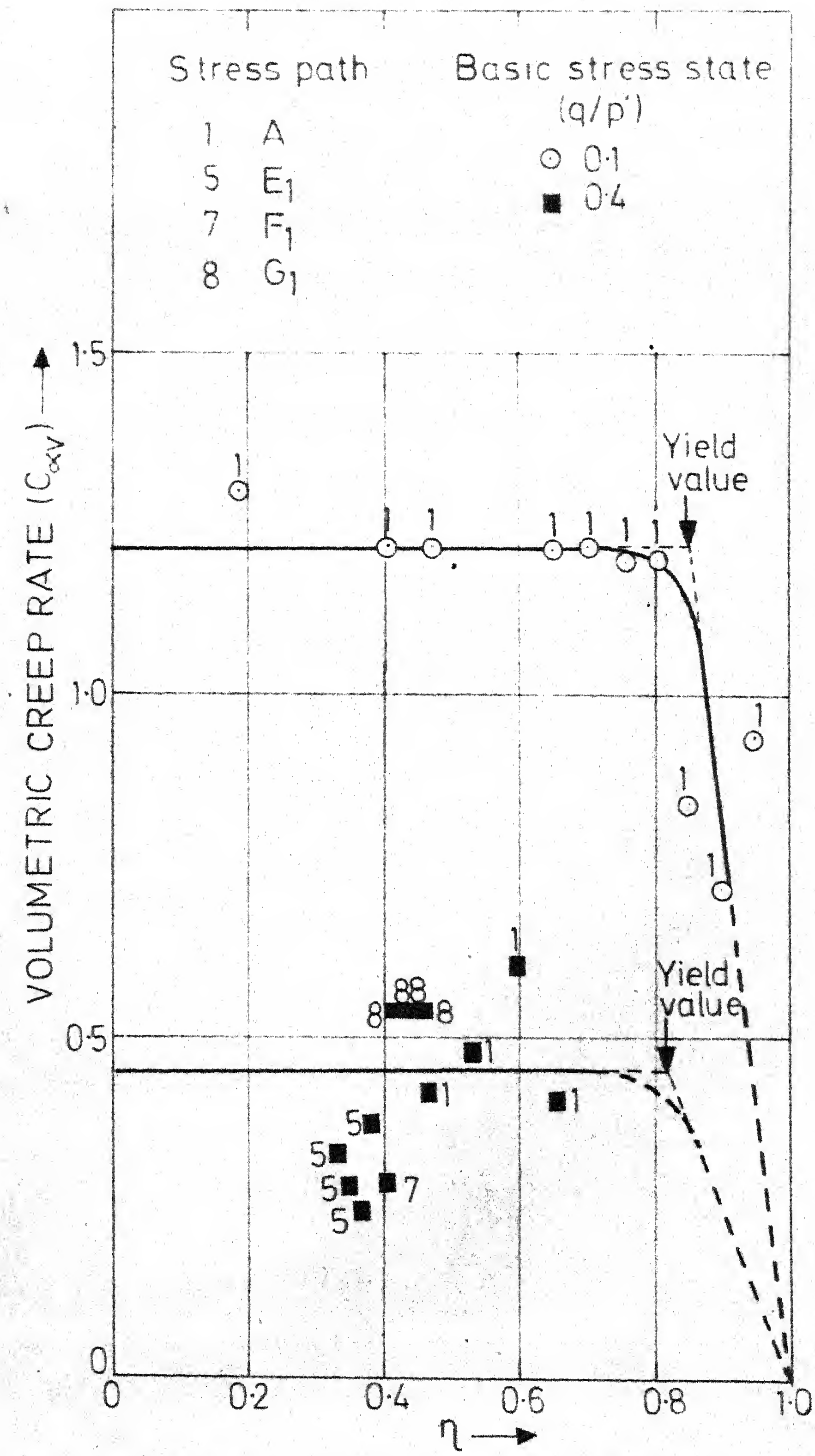
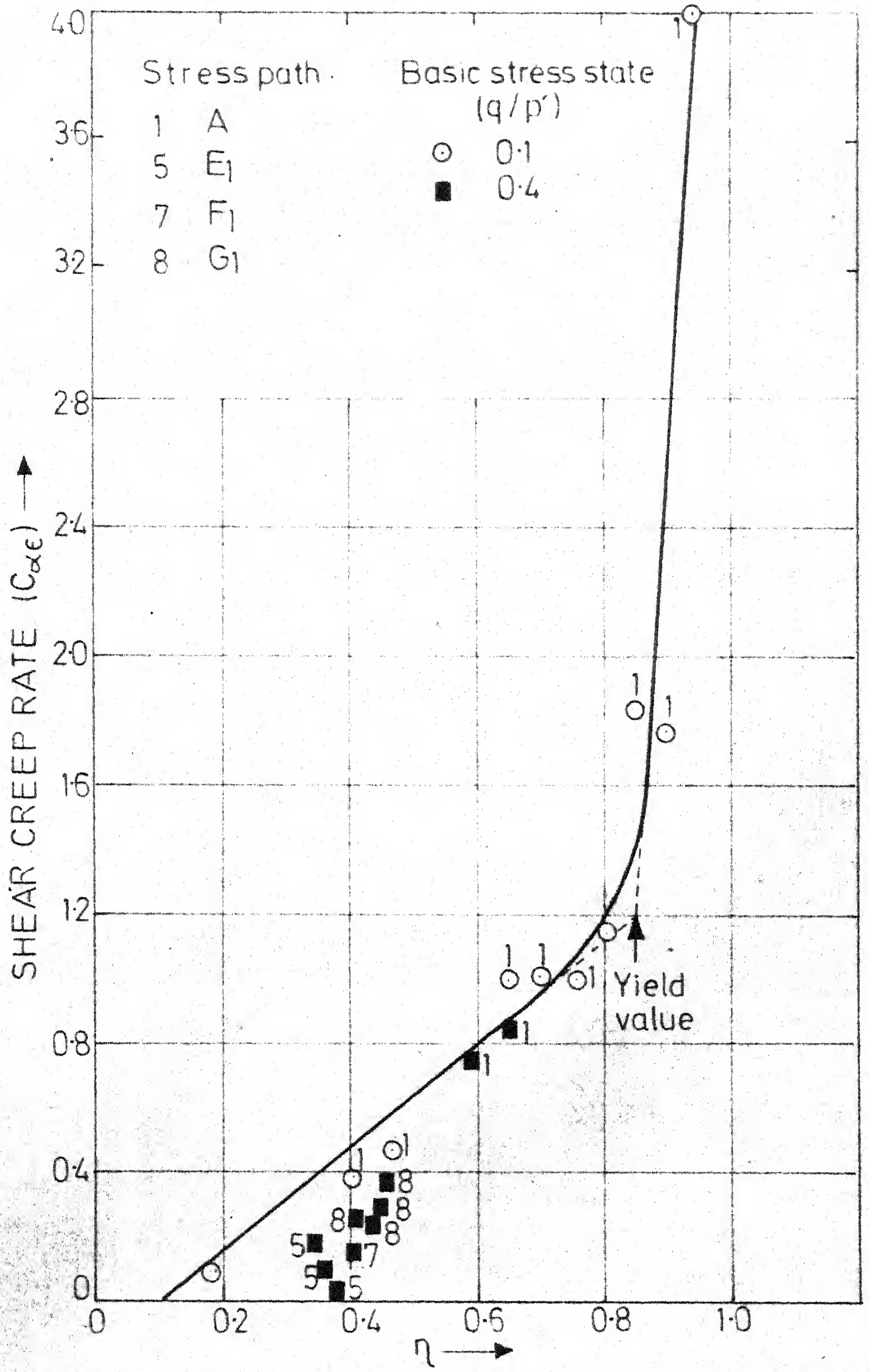


FIG. 5·32 (a) RELATIONSHIP BETWEEN STRESS RATIO (η) AND VOLUMETRIC CREEP RATE ($C_{\alpha v}$) FOR NCC



FIG·5·32 (b) RELATIONSHIP BETWEEN STRESS RATIO(η) AND SHEAR CREEP RATE ($C_{\alpha\epsilon}$) FOR N.C.C.

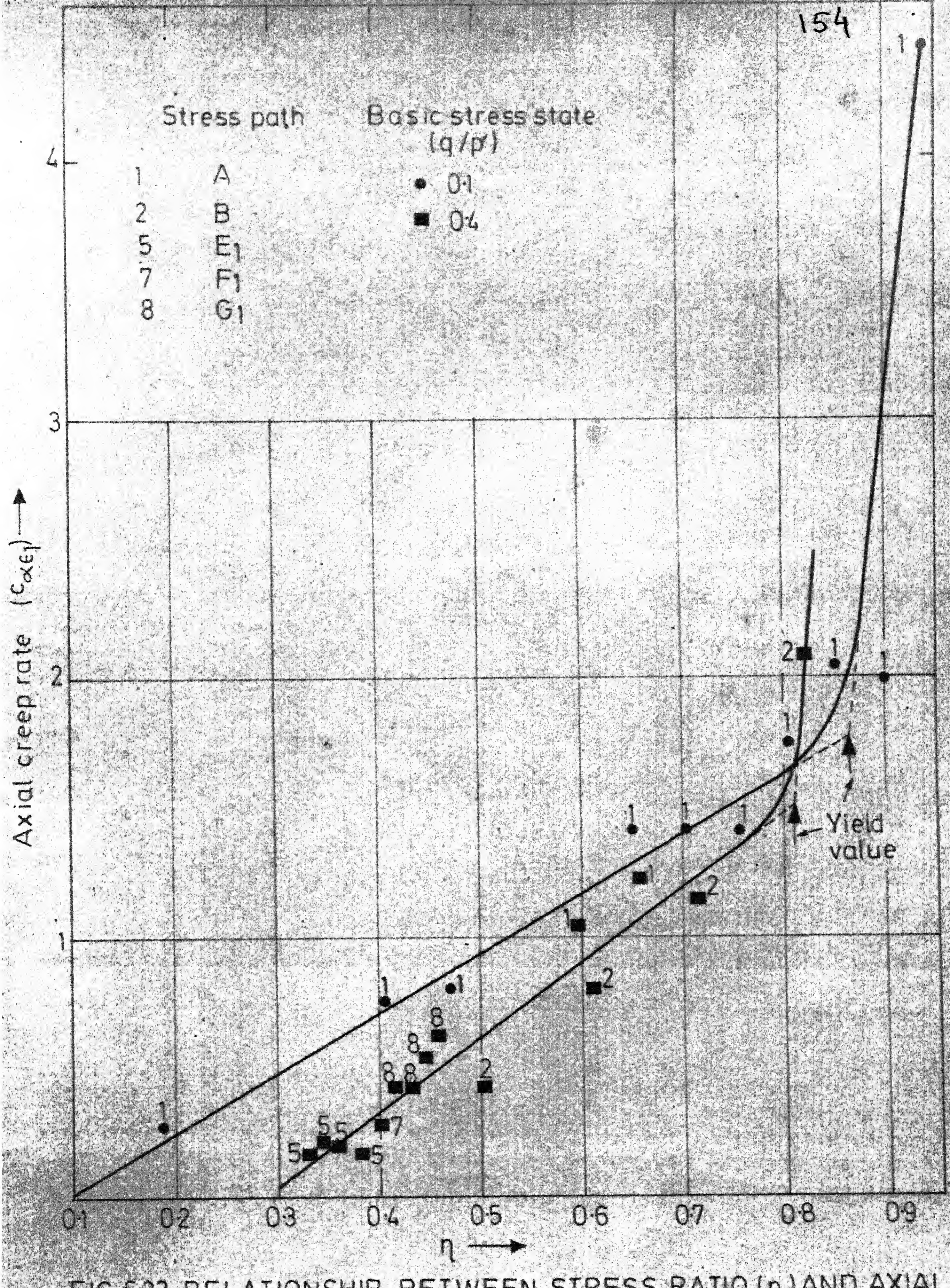


FIG 5·33 RELATIONSHIP BETWEEN STRESS RATIO (η) AND AXIAL CREEP RATE ($C_{\alpha\epsilon_1}$) FOR NCC

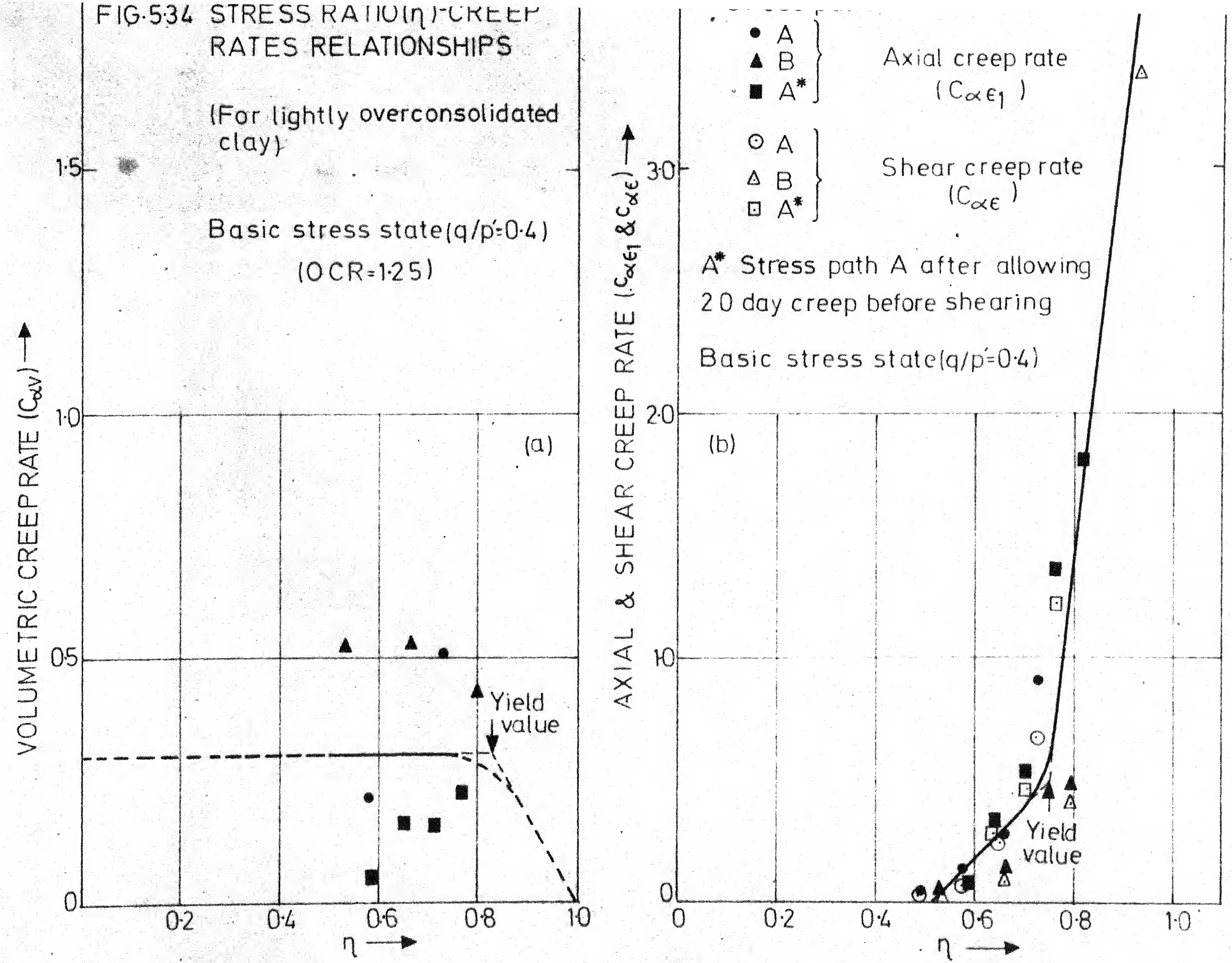
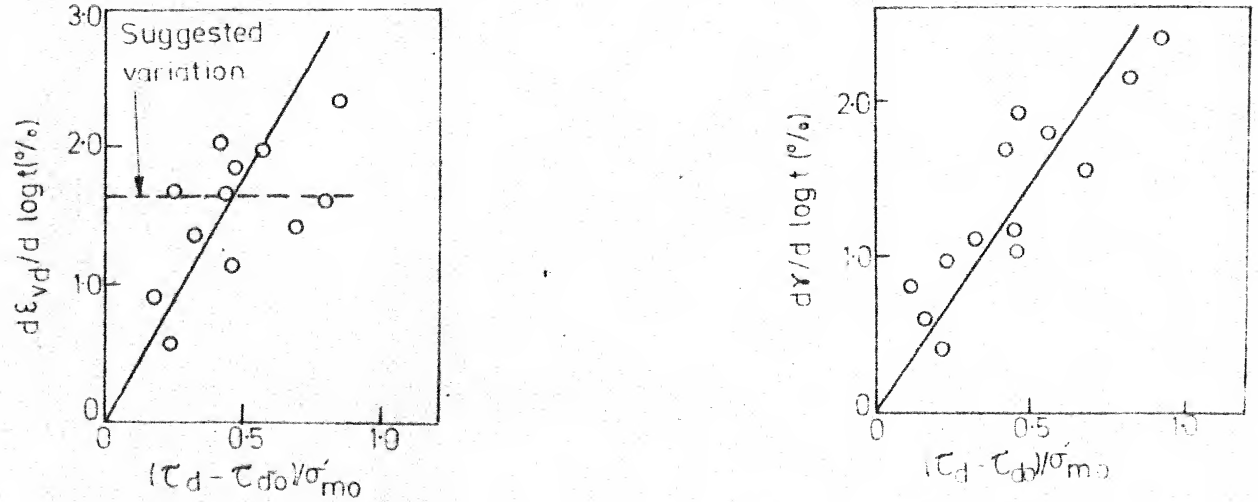
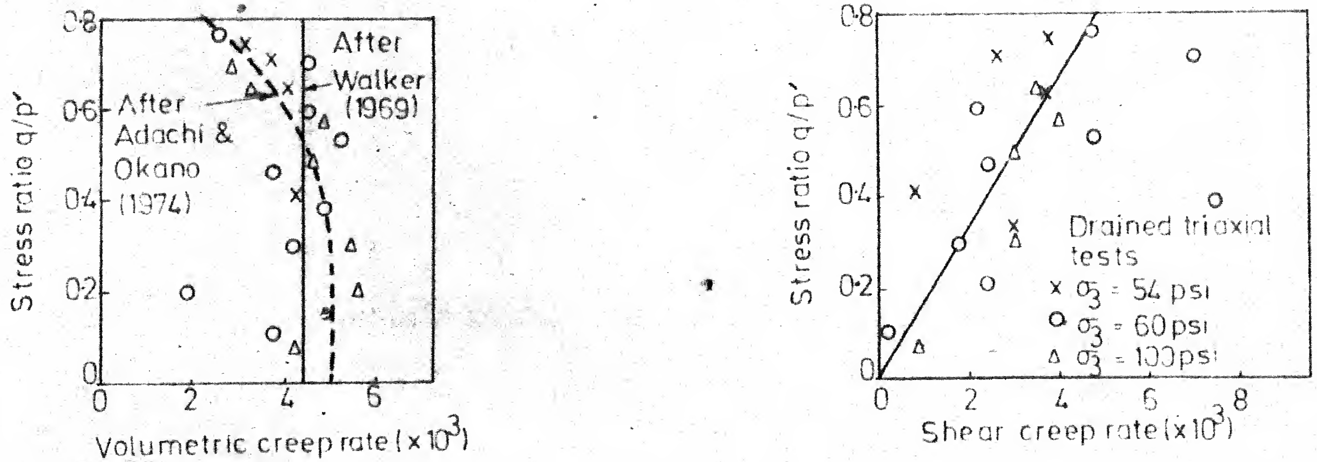


FIG.5.34 STRESS RATIO(η)-CREEP RATES RELATIONSHIPS



FIG·5·35(b) DRAINED P-CONSTANT CREEP TESTS
(AFTER YAMANOUCHI AND YASUHARA 1975)



FIG·5·35(a) DRAINED CONVENTIONAL CREEP TESTS
(AFTER WALKER 1969)

CHAPTER 6

PREDICTION OF THE OBSERVED STRESS-STRAIN-TIME BEHAVIOUR

6.1. GENERAL

The behavioural aspects of stress-strain and creep were discussed in chapters 4 and 5 respectively. It was shown that theory of elasticity cannot handle the stress path dependent, dilatant behaviour of soils. A model is proposed in this chapter, which is able to predict the observed stress-strain behaviour of most of the stress paths studied in this investigation. It is also shown that the proposed model can predict the anisotropic consolidation test results, value of k_o, state boundary surface governing volume change behaviour during shear & also undrained test results for a k_o-consolidated sample. The predictions from various available models based on theory of plasticity concepts and empirical approaches are also compared. It is shown that the drained creep rates for some of the stress paths can also be predicted. Finally, the parameters m, $\bar{\alpha}$, A for the Berkeley rate process model are evaluated from the drained creep tests for various stress paths and the suitability of this model to predict the drained creep rates is discussed.

6.2 PROPOSED MODEL TO PREDICT THE STRESS-STRAIN BEHAVIOUR

From the work of various investigators, and also as observed from the results of this investigation, it is clear that the triaxial behaviour of saturated clays is mainly dependent on the mean normal stress P' and the shear stress q. It is now suggested, that the stress-strain behaviour of a saturated clay along any stress path, may be predicted from the results of two basic tests:

(a) a consolidation test (q-constant) starting from a k_o-line.

(b) a pure shear test (P-constant) starting from a k_o-line.

As has been observed that both the volumetric and the shear strains are obtained in each of the above two tests, it is clear that decoupling of these effects, due to changes in P & q assumed in theory of elasticity, is not correct. Any theory, which seeks to predict changes in volumetric & shear strains resulting from changes in P and q, should, therefore, consider this coupled behaviour as suggested by Scott & Ko (1969). In the proposed model, this has been attempted.

The changes in volumetric & shear strains, as a result of the application of a probe (dp, dq) under drained conditions, may be expressed as :

$$dv = \frac{\partial v}{\partial p} dp + \frac{\partial v}{\partial q} dq \tag{6.1}$$

$$d\varepsilon = \frac{\partial \varepsilon}{\partial p}\, dp + \frac{\partial \varepsilon}{\partial q}\, dq \qquad (6.2)$$

where, dv and $d\varepsilon$ are the changes in volumetric and shear strains respectively during the application of a probe. $\partial v/\partial p$ and $\partial \varepsilon/\partial p$ are the parameters, which can be determined from a q-constant test, whereas the parameters $\partial v/\partial q$ and $\partial \varepsilon/\partial q$ have to be obtained from a p-constant test. The assumption is that the total effect of any loading increment comprise the separate effects of the corresponding increments of p & q. Eq. 6.1 and 6.2 are general and can handle any stress path. Most important consideration is, however, evaluation of the relevant parameters to be used in these equations.

6.2.1 Evaluation of the Parameters

The parameters $\partial v/\partial p$, $\partial \varepsilon/\partial p$, $\partial v/\partial q$ and $\partial \varepsilon/\partial q$ in Eqs. (6.1) and (6.2) have to be obtained from relevant p & q constant tests depending upon the stress path. Table 6.1 shows the suggested procedure to be adopted. For the evaluation of these parameters, test results of the four basic stress paths E_1, B, E and B-unloading are plotted and discussed subsequently. As the basic stress state $q/p' = 0.4$ is near k_o of this clay, the prediction of strains has been attempted for the tests from this basic stress state. However, some test results from the basic stress state of $q/p' = 0.2$ have also been indicated for comparison.

As the probes investigated were relatively small (twenty percent of the basic stress state only), these parameters have been evaluated for this clay on the basis of the available data. As shown subsequently, the values of the parameters as chosen, appear to be reasonable, however, for a more accurate evaluation, the test results to failure are desirable.

6.2.1.1. Parameter Determination from E_1 Test

Fig. 6.1 shows the volumetric and axial strains plotted against ln P. The straight line relationships as shown appear to be reasonably fitting the limited data. The slopes of these lines are $\alpha_1 = 0.076$ and $\alpha_3 = 0.02$. From these tests results, the following relationships are obtained:

$$\frac{\partial v}{\partial p} = \frac{\alpha_1}{p} \quad \text{and} \quad \frac{\partial \varepsilon}{\partial p} = \frac{\alpha_3}{p} \qquad (6.3)$$

6.2.1.2 Parameter Determination From E-Test

As no test was available from the basic stress state of $q/p' = 0.4$, results of E-test from the basic stress state of $q/P' = 0.2$ are plotted and used for predicting the strains of tests conducted from the other basic stress state. The constants, $\alpha'_1 = 0.0064$ & $\alpha'_3 = -0.0036$ are obtained as shown in Fig. 6.2. α'_3 is negative, as for the small increment, shear strains are positive even though the pressure P' is decreasing. As discussed

earlier, this may be due to the tendency of the material to creep and also the magnitude of small steps adopted in this investigation for all the stress paths.

Following relationships are obtained from this test:

$$\frac{\partial v}{\partial p} = \alpha_1'/p \quad \text{and} \quad \frac{\partial \varepsilon}{\partial p} = - \alpha_3'/p \qquad (6.4)$$

6.2.1.3. Parameter Determination From B-Test

The stress-strain curve for this stress path is highly non-linear and in order to account for this non-linearity, the test results have been plotted in an unconventional manner shown in Figs. 6.3 and 6.4, as suggested by Wroth (1968). The advantage of this plot is that the straight lines obtained in $\ln (q_f/q_f-q)$ versus shear strain and q/q_f versus volumetric strain plots (q_f being the shear stress at failure), have the same slope D', with D' being the value of the total volumetric strain experienced by the sample during the test. D' is 0.045 for normally consolidated and 0.0072 for lightly overconsolidated clay samples (Fig. 6.3). Test results on isotropically consolidated samples of this clay are shown in Fig. 6.4. D' for this test is 0.036. It can be seen from these figures that Worth's plot can represent the entire stress-strain-volume change curve for B stress path by means of linear relationships. The differences in D' values in Figs. 6.3 and 6.4 could either be due to the different stress history of the samples

(i.e., isotorpic/k_o-consolidation) or may as well be due to the differences in samples used in the two investigations. Following relationships are obtained for this test.

$$\frac{\partial v}{\partial q} = \frac{D'}{q_f} \quad \text{and} \quad \frac{\partial \varepsilon}{\partial q} = \frac{D'}{q_f - q} \tag{6.5}$$

6.2.1.4 Parameter Determination From B-Unloading Test

The corresponding relationships for unloading along the stress path B are not so easy to evaluate due to the inaccuracies involved in the measurements of small strains. The test results of unloading along the stress path B have indicated some recovery in shear strains with continuing increase of compressive volumetric strains (Refer Figs. 4.23 & 4.24).

For the loading part of the shear stress (q)-shear strain (ε) curve, the slope at any point of this curve is given by Eq. 6.5

$$\text{i.e.,} \quad \frac{\partial q}{\partial \varepsilon} = \frac{q_f - q}{D'} \tag{6.6}$$

For the unloading part of the curve, the slope at any point decreases as q decreases. Following Wroth (1968), the corresponding relationship may be assumed as:

$$\frac{\partial q}{\partial \varepsilon} = \frac{q_f + q}{C'} \tag{6.7}$$

where, C' is a constant.

For the volumetric strains, it is assumed that the relationship given by Eq. (6.5) holds good.

i.e.,
$$\frac{\partial v}{\partial q} = -\frac{B'}{q_f} \tag{6.8}$$

where B' is a constant.

Eqns. (6.7) and (6.8) on integration give

$$\varepsilon - \varepsilon_o = C' \ln \frac{q_f + q}{2q_f} = -C' \ln \left(\frac{2q_f}{q_f + q}\right) \tag{6.9}$$

and
$$v - v_o = -\frac{B'}{q_f}(q - q_f) = \frac{B'}{q_f}(q_f - q) \tag{6.10}$$

Assuming the relationships given by Eqns. 6.9 and 6.10 to hold good for the unloading behaviour along B stress path for this clay, plots indicated in Figs. 6.5 and 6.6 should then give straight lines of slopes $1/c$, and $-1/B'$ (for normally & lightly overconsolidated samples). As the unloading along B stress path for the normally consolidated sample was not carried out below the k_o-line, the values of $C'=0.005$ and $B' = 0.0024$, as given in Fig. 6.5 will be used in the predictions.

For B-unloading test, following relations are obtained:

$$\frac{\partial v}{\partial q} = -\frac{B'}{q_f} \quad \text{and} \quad \frac{\partial \varepsilon}{\partial q} = \frac{C'}{q_f + q} \tag{6.11}$$

It is seen from Figs. 6.5 and 6.6, that the volumetric strains are better represented than the shear strains by the relationships given by Eqns. 6.9 and 6.10. However, as these equations are simple and can be conveniently used, for want of better expressions to represent the unloading behaviour, they have been adopted here (Wroth, 1968).

Equations 6.1 and 6.2, used in conjunction with Eqns. 6.3 through 6.11, give the following expressions, which can be used to predict the stress-strain-volume change behaviour for practically all the stress paths.

For stress paths with ΔP and Δq increasing :

$$dv = \alpha_1 \frac{dP}{P} + \frac{D'}{q_f} dq \tag{6.12}$$

$$d\varepsilon = \alpha_3 \frac{dP}{P} + \frac{D'}{q_f - q} dq \tag{6.13}$$

For stress paths with ΔP decreasing and Δq increasing:

$$dv = \alpha_1' \frac{dP}{P} + \frac{D'}{q_f} dq \tag{6.14}$$

$$d\varepsilon = \alpha_3' \frac{dP}{P} + \frac{D'}{q_f - q} dq \tag{6.15}$$

For stress paths with ΔP and Δq decreasing:

$$dv = \alpha_1' \frac{dP}{P} - \frac{B'}{q_f} dq \tag{6.16}$$

$$d\varepsilon = \alpha_3' \frac{dP}{P} + \frac{C'}{q_f + q}\, dq \qquad (6.17)$$

and for stress paths with ΔP increasing and Δq decreasing:

$$dv = \alpha_1 \frac{dP}{P} - \frac{B'}{q_f}\, dq \qquad (6.18)$$

$$d\varepsilon = \alpha_3 \frac{dP}{P} + \frac{C'}{q_f + q}\, dq \qquad (6.19)$$

Eqns. 6.12 through 6.17 have been made use of to predict the stress-strain behaviour of stress paths A_1, G_1, F_1, C, D, F & G. As no tests are available in the sector $0 < \theta < 60°$, the predictions from Eqns. 6.18 and 6.19 could not be checked.

6.2.2. Comparison of the Predictions by the Proposed Model with other Methods

Figs. 6.7 and 6.8 (a) show the predictions of the loading stress paths A, G_1 and F_1 (indirect). They are obtained using eqns. 6.12 and 6.13. Predictions by the two Cambridge models* and Wroth's model (1968) are also given for comparison. The suitability of the proposed model to predict the observed stress-strain behaviour for the stress paths A and G_1 is evident.

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* Cam Clay, Burland (1965), Roscoe & Burland (1968) models are hereafter referred to as Cambridge models.

The Cam Clay overpredicts the shear strains considerably. Burland (1965) also predicts higher shear strains. At higher stress ratios, the predictions by Burland are rather bad. Wroth's model gives the same order of magnitudes of shear strains as measured for the stress path G_1, but predicts higher shear strains, for the stress path A. Volumetric strain predictions are in general comparable with experimental data except for Cam clay.

The prediction of strains for F_1 stress path cannot be accurately assessed, since this is an indirect test (refer special test 1) on a sample having a different stress history. It is evident from Fig. 6.8 (a), that the strains as predicted for this stress path would have been close to the measured values, had F_1 test been conducted immediately after the rest period following anisotropic consolidation.

The predictions by different models for the other loading stress paths B and E_1 are shown in Figs. 6.9 (a) and (b) respectively. As the proposed model is derived from these two basic stress paths, its predictions have not been included. For the same reason, the prediction of B stress path by Wroth's model is not shown. Strains for the stress path E_1 are predicted quite well by Wroth's model, whereas the Cambridge models predict results which differ a lot from the observed values. For stress path B, although the predictions by Burland are much better than those from Cam Clay,

yet it overpredicts the measured shear strains.

Fig. 6.10 compares the predictions of the results of the only complete test (conventional drained test) carried out in this investigation. The proposed model shows very good agreement, even though this basic stress state is close to the isotropic state. Burland's predictions are good upto the stress ratio of 0.6, beyond which the shear strains are overpredicted. Wroth's model in general predicts larger shear strains. Cam clay predictions are much too large. Volumetric strains are predicted quite well by these models for this test.

For the stress paths C and D (refer Table 6.1) predictions have been made from Eqns. 6.14 and 6.15 using $D' = 0.0072$. The model predicts shear strains for the stress path C very well, whereas, the predictions for stress path D are far from satisfactory [Figs. 6.11 (a) and (b)]. Volumetric strain predictions are also not so good. Cambridge models are applicable for "wet" clays only and as such these cannot predict the shear strains for these stress paths. The volumetric strains, in general for all unloading probes, are obtained by using 'K' values only. These predictions are considerably larger than those by the proposed model and have not been shown. Wroth's model, although it overpredicts the strain considerably, atleast shows the observed trends.

Fig. 6.8(b) shows the predictions for the stress path G (Eqns. 6.16 and 6.17 have been used for G & F stress paths). The predictions compare well with the observed strains. Wroth's model does not adequately describe the observed behaviour. The predictions for the stress path F, shown in figs. 6.12 and 6.13, although not very good, yet show the right trend. The effect of the magnitude of the probe (as discussed in Chapter 6) is also indicated. Full probe in one shot, gives much larger strains than the corresponding probe in four equal increments.

The discrepancy between the predictions and observations for the unloading stress paths D, F, and G may be due to: (i) non-availability of the complete test data from proper tests (B-unloading test should have started from the anisotropic line, unlike the B test for OCR = 1.25 used and E stress path test from q/P' = 0.4 stress state should have been used). (ii) the combined effects of creep and unloading, (for unloading stress paths G and F, where both P and q decrease, the response in the initial stages is delayed because of the overriding influence of creep due to the small unloading increment used). Refer Figs. 6.12 and 6.13. Poulos (1964) makes similar observations for materials exhibiting creep. It is quite likely that the single parameter representation of the behaviour for such stress paths in this model, may not be able to account for the observed trends in Figs. 6.12 and 6.13.

In order to check these possibilities, data from appropriate tests to failure is needed.

6.2.2.1 Discussions of the Predictions

Based on the parameters, as evaluated from the limited test data available, the proposed model shows excellent agreement between observed and predicted strains for all the loading stress paths.

The model also predicts the behaviour rather well for unloading stress paths such as C, G and F and it is likely that the test data for the stress path D may be in error.

The Cam clay model, as is now well known, considerably over-predicts the strains. Burland's (1965) model also cannot predict the strains accurately for stress paths E_1, G_1, A and B for K_o-consolidated samples. This method overpredicts the shear strains considerably. The modifications suggested by Roscoe and Burland (1968) would predict still larger values of shear strains and as such is of not much help in this case. Newland (1973) has shown that for E_1 tests conducted from various anisotropic consolidation lines, the predictions by the Cambridge models are rather bad. Fig. 6.14 (b) shows this comparison. Lewin and Burland (1970) surprisingly never compared, the test results of their detailed investigation with the Cambridge models. Shear strains were predicted from various yield surfaces obtained from three full

tests. Fig. 6.14 (a) shows the details of the comparison made. It is clear that the prediction of shear strains from various yield surfaces also donot compare well with the measured values, although, they had specially selected such a material for this study which showed very little creep, so that the results are not affected by it.

Results of the present investigation, along with the findings of Newland (1973) and Lewin & Burland (1970), confirm that the Cambridge models based on constant values of λ, K and M cannot predict the triaxial behaviour of the K_o-consolidated samples. Incidentally, the assumption in these models regarding constant values of λ and K, is probably a gross simplification. Newland's results (shown in fig. 4.3a) and the results of the present investigation (shown in Fig. 4.2) clearly indicate that λ decreases as η approaches M. Newland has further shown dependence of K/λ and M on pressure. Varadarajan (1973) & Rahman (1972) have shown dependence of M on the pressure for the clay used in this investigation. These variations in λ, K and M with the stress ratio η and the confining pressure may explain some of the difficulties associated with these models. A detailed investigation regarding this variation in λ and K particularly, is urgently needed. Newland (1973) rightly comments on the predictions by the Cambridge models that "any theory which does not correctly predict the stress-strain curves for

the two fundamental phases of undrained shear and consolidation under constant $q(E_1$ test), will not in general successfully predict the strains for the combination of these two phases". The present model attempts to meet these requirements. Newland further points out that the agreement between the predicted and the measured strains obtained by him for the anisotropic consolidation test results by Burland's model, should be considered fortuitous for reasons mentioned above.

The reason why the Cambridge models were successfully shown as being able to predict the observed behaviour (See Roscoe and Burland, 1968); is due to the fact that the verification of the predictions from these theories, was mainly attempted from test results conducted on isotropically consolidated samples (Parry, 1956; Loudon, 1967; Schofield and Wroth, 1968). Further proof of this can be had by comparing the predictions of the drained test results on isotropically consolidated samples of this clay by various models. This is shown in Figs. 6.15 (a) & (b). Dotted curves in these figures are due to modifications suggested by Roscoe and Burland (1968).

The relationship given in Fig. 6.16 has been made use of to obtain Roscoe & Burland's predictions. Roscoe & Burland's model predicts the strains for both the stress paths very well. Wroth's model underpredicts the shear strains at higher stress level. No prediction by the proposed model is shown in Fig. 6.15. However, it is clear that it will predict the smallest values of shear strains for the same stress ratio η. This is to be expected, since the model parameters have been evaluated from tests on K_o-consolidated samples. Parry's (1956) B stress path test data on isotropically consolidated Weald clay has been compared with the predictions by the Cambridge models in Fig. 6.17. It can be seen

that Burland's model predicts shear strains well upto $\eta = 0.65$. In this case also, the modification by Roscoe and Burland (1968) beyond $\eta = 0.65$ will result in overprediction of shear strains, (similar to that observed in Fig. 6.15b).

From the foregoing discussions, it is evident that the Cambridge stress-strain models predict the volume change response quite adequately both for isotropically and anisotropically consolidated samples. The predictions of shear strains, however, are in error for all stress path tests with k_o-consolidation history and even for some stress path tests, like B (at higher η-values) conducted on isotropically consolidated samples. The errors in the predictions of the shear strains by the Cambridge models are further due to the fact that the associated flow rule of the theory of plasticity, used in predicting the plastic strain increment vectors, is not strictly valid for soils. This is because the direction of the plastic strain increment vector is dependent on the direction of the stress increment vector for clays. Fig. 6.18 shows the stress probes along with the plastic strain increment vectors for the two basic stress states, as obtained in this investigation and confirms the rotation of the strain increment vector with stress path. This has also been reported by Lelievre (1967) and Lewin & Burland (1970). Lewin (1971), after his detailed experimental study described in Lewin and Burland (1970), rightly states in this paper that "unless the rotation of the strain increment vector is allowed for then serious

difficulties arise when predicting the strains that are caused by imposed stresses". The serious difficulties in the strain prediction as stated by Lewin, obviously point to the inability of the Cambridge models to predict the results of the tests along various stress paths conducted by him on anisotropically consolidated samples (as described in Lewin & Burland, 1970). Lewin (1973) has also very well demonstrated the effect of the stress history on the plastic potential. He makes use of various anisotropic consolidation tests to obtain a plastic potential, which he combines with the empirical approach of Roscoe and Poorooshasb (1963) to describe a method claimed by him as a "useful advance" over Roscoe & Burland model. However, the predictions of the results of Lewin & Burland's (1970) study, have not been compared with this new approach.

In the end it is pointed out that the empirical approach as proposed in this investigation appears to be quite appropriate at the present moment (for example, see Wroth, 1968 and Lewin, 1975). The suggested model is extremely simple both in respect of its formulation and evaluation of the relevant parameters. Only four basic tests are required to predict the behaviour of all stress paths in the triaxial stress space (Refer Table 6.1). However, it is quite clear that more detailed work along the suggested lines is needed for a variety of natural clays, to evaluate the validity of the predictive power of this model. No comparison of the predictions

by this model, of the test results on other soils, could be attempted, due to the lack of data for the two basic tests (i.e., B & E_1) required to evaluate the parameters needed. In comparison to other models discussed earlier, it has the versatility to account for both the relevant field stress history and the stress path dependence. The same approach could be extended for predicting the behaviour under more general three-dimensional stress conditions, making use of a true triaxial test apparatus.

The model can also be readily used to predict the settlement of footings on clays, using the approach suggested by Lambe (1964, 1967) and Davis & Poulos (1968) as follows:

From Eqs. 6.12 and 6.13, the axial strain increment can be written as:

$$d\varepsilon_1 = d\varepsilon + \frac{1}{3} dv \qquad (6.20)$$

The axial strain increment obtained from Eq. 6.20 is then summed up along the depth as follows to obtain the total settlement ε_1:

$$\varepsilon_1 = \int_0^H d\varepsilon_1 \, dz \qquad (6.21)$$

where H is the depth of the clay layer.

From the comparisons of volumetric and shear strain predictions by various models as discussed earlier, it is obvious that the proposed model will predict less axial strains for the soil under investigation,

compared to the method suggested by Burland (1972). (Lambe, 1965 shows that the refined stress path method will predict far less settlement than by oedometer and other methods). Of course, the proposed method has the same limitation as the methods of Lambe and Burland in respect of effect of initial yield on settlement.

6.2.3 Prediction of the Anisotropic Consolidation Test Results by the Proposed and other Models.

By definition,

$$q = \eta P \tag{6.22}$$

For η-constant tests, Eq. 6.22 can be written as

$$dq = \eta \, dP \tag{6.23}$$

Since for anisotropic consolidation,

$$q_f = MP_f = MP \tag{6.24}$$

Substituting Eqs. 6.23, 6.24 in Eqs. 6.12 and 6.13, following relations are obtained

$$\dot{v} = [\alpha_1 + \frac{D'\eta}{M}] \frac{dP}{P} \tag{6.25}$$

and $$\dot{\varepsilon} = [\alpha_3 + \frac{D'\eta}{M-\eta}] \frac{dP}{P} \tag{6.26}$$

The flow rule is obtained from Eqs. (6.25) and (6.26) as follows:

$$\frac{\dot{\varepsilon}}{\dot{v}} = \frac{\alpha_3 + \frac{D'\eta}{M-\eta}}{\alpha_1 + \frac{D'\eta}{M}} \tag{6.27}$$

or
$$\frac{\dot{\varepsilon}}{\dot{v}} = \frac{\alpha_3/\alpha_1 + D'/\alpha_1 \ \frac{\eta}{M-\eta}}{1 + D'/\alpha_1 \ \frac{\eta}{M}} \tag{6.28}$$

With $\alpha_3 = 0.02$, $\alpha_1 = 0.076$; $D' = 0.045$;

$$\alpha_3/\alpha_1 = 0.263 \quad \text{and} \quad D'/\alpha_1 = 0.591 \tag{6.29}$$

Substituting Eq. 6.29 and $M = 1$ in Eq. 6.28 gives:

$$\frac{\dot{\varepsilon}}{\dot{v}} = \frac{0.263 + 0.591 \ \eta/(1-\eta)}{1 + 0.591\ \eta} \tag{6.30}$$

Eq. 6.30 can predict the total strain increment ratio $\dot{\varepsilon}/\dot{v}$ for anisotropic consolidation test for various η-values. Fig. 6.19 shows the prediction from Eq. 6.30 along with the predictions by the two Cambridge models. At $\eta = 0$, Eq. 6.28 reduces to $\dot{\varepsilon}/\dot{v} = 0.263$. It is evident that $\dot{\varepsilon}/\dot{v}$ cannot be equal to 0.263 for the isotropic tests. The exact value ofcourse depends on the parameters α_1, α_3. However, a small positive value of the ratio $\dot{\varepsilon}/\dot{v}$ has been observed during isotropic consolidation partly due to the anisotropy induced at the time of sample formation (Simons & Som,1969 and Lewin, 1971 & 1973). Experimental points have also been marked for comparison. The predictions by proposed models lie, in general, close to those by Roscoe & Burland's(1968). In fact the predicted curve almost fits the

Eq. $\dot{\varepsilon}/\dot{v} = \frac{1.5\eta}{M^2-\eta^2}$, which is similar to $\dot{\varepsilon}/\dot{v} = \frac{2\eta}{M^2-\eta^2}$ proposed by Roscoe & Burland (1968).

Fig. 6.20 shows the anisotropic consolidation test results on other soils. The predicted curve, in general, agrees with the observed behaviour i.e., when $\eta \rightarrow M$; $\dot{\varepsilon}/\dot{v} \rightarrow \infty$. But, Namy (1968) predicts a linear relationship between η and $\dot{\varepsilon}/\dot{v}$. The test results of Namy, however, do indicate a tendency to follow the behaviour observed by many investigators on different clays, as shown in this figure. The general trend of $\dot{\varepsilon}/\dot{v} \rightarrow \infty$ as $\eta \rightarrow M$ appears more logical considering that normally consolidated clays at higher stress ratios show much larger shear strains and gradually flow at constant volume as critical state is approached, which evidently shows that $\dot{\varepsilon}/\dot{v}$ should approach ∞ as η approaches M. The comparison of the predicted behaviour during anisotropic consolidation for this clay with that observed in general by others show that the model can predict the behaviour very well.

6.2.3.1. Prediction of K_o by the Proposed Model

The predicted value of K_o is shown in Fig. 6.19 corresponding to a ratio $\dot{\varepsilon}/\dot{v} = 2/3$. The value of K_o is 0.625, as compared to the measured value of 0.685. Eq. 6.28 can be used to get a relationship between K_o and ϕ' as follows :

Substituting $\dot{\varepsilon}/\dot{v} = \frac{2}{3}$ in Eq. 6.29 and simplifying

$$\eta^2 + \frac{M\alpha_1}{D'}(1+0.5\ D'/\alpha_1 - 1.5\ \alpha_3/\alpha_1)\eta + (1.5\ \alpha_3/\alpha_1 - 1)M^2\ \alpha_1/D' = 0 \qquad (6.31)$$

Eq. 6.31 is similar to the relationship of Roscoe and Burland (1968) given by:

$$\eta^2 + 3(1 - k/\lambda)\eta - M^2 = 0 \qquad (6.32)$$

From Eq. 6.31, for various combinations of M, D'/α_1 and α_3/α_1 ; K_o can be predicted in a manner similar to the curves given by Roscoe and Burland (1968) for various combinations of K/λ and M. However, to obtain a simplified relationship between K_o and ϕ', substitution of $D'/\alpha_1 = 0.591$ and $\alpha_3/\alpha_1 = 0.263$ in Eq. 6.31 gives

$$\eta^2 + 1.525\ M\eta - 1.025\ M^2 = 0 \qquad (6.33)$$

Substituting $M = \frac{6\ \sin\ \phi'}{3 - \sin\ \phi'}$ and $K_o = \frac{3 - \eta_{k_o}}{3 + 2\eta_{k_o}}$ in Eq. (6.32) and then simplifying; the following relationship is obtained.

$$K_o = \frac{1 - 0.661\ \sin\ \phi'}{1 + 0.344\ \sin\ \phi'} \qquad (6.34)$$

Eq. 6.34 is plotted in Fig. 6.21 to give the variation of K_o with ϕ'. Predictions by Jaky (1944), Wroth (1968), Lewin (1970, 1971) and the experimental values of K_o obtained in the laboratory by a number of investigators are also shown in this figure. The predicted curve is close to many experimentally observed values for clays.

6.2.3.2 Prediction of the State Boundary Surface.

Using the flow rule as given by Eq. 6.28 and following Roscoe & Burland (1968), the equation for the state boundary surface is obtained as follows:

$$\ln \frac{P_o}{P} = \frac{1}{2} \ln [a\eta^2 + b\eta + d]_{\eta_o}^{\eta} + 2.34 \ln \left[\frac{2a\eta - 1.62}{2a\eta - 0.77}\right]_{\eta_o}^{\eta} \qquad (6.35)$$

where: a, b, d are constants given by:

$a = (-AM + BM - B)$,

$b = (AM^2 - M + BM)$,

$d = M^2$,

where,

$A = \alpha_3/\alpha_1$,

and $B = D'/\alpha_1$.

Fig. 6.22 shows the predictions of Eq. 6.35. Roscoe & Burland's (1968) predictions for $M = 1$ and 1.1 are shown separately. The experimental data of this investigation and also the data on isotropically consolidated samples of this clay are presented. The fact that the predicted state boundary surface is close to Roscoe and Burland's model, shows that the volumetric strain predictions by these models would be very similar. This was observed in the discussion given earlier in this chapter.

Also as pointed out earlier, the relation $\dot{\varepsilon}/\dot{v} = \frac{1.5\eta}{M^2 - \eta^2}$ used here is very close to the relation used by Roscoe and Burland.

The test results of A and B stress paths rise up and ultimately join the state boundary surface as shown, where it starts yielding. Similar behaviour is observed for lightly overconsolidated samples both for isotropically and anisotropically consolidated clays. The fact that clays show clear yielding is illustrated from the limited data in this investigation in Figs. 6.23a and 6.23b. Stress path tests (A, B and C) conducted on lightly overconsolidated clay samples show yielding clearly and trace of the volumetric yield locus for lightly overconsolidated clay is given (Fig. 6.23b). This substantiates the findings of Parry and Nadarajah (1974) and Mitchell (1970). This also confirms the broad concepts presented by the Cambridge group on the state boundary surface.

6.2.3.3 Other Predictions

The proposed model could be used to predict the undrained test results. Putting $dV = 0$ in Eq. 6.12, would predict the pore pressure response and Eq. 6.13 would then predict the stress-strain curve. This has not been worked out, as no undrained test for K_o-consolidated samples was conducted.

Also K_o-unloading behaviour could be predicted using Eqs. 6.16 and 6.17, by putting the condition for K_o (i.e.,$d\varepsilon/dv = 2/3$), as suggested by Wroth (1968).

6.3 PROPOSED MODEL TO PREDICT THE DRAINED CREEP RATES

The drained creep test results, presented in Chapter 5, show that the creep rates are dependent on the stress path and the stress ratio. It is suggested that the logarithmic creep rates (C_α) for various stress paths may be predicted from the creep rates of two basic tests: (i) P-constant creep test and (ii) q-constant creep test; the assumption being that the effect of any increment on creep rates consists of the separate effects of the corresponding increments of P & q. Thus, the axial creep rate may be given as:

$$dC_{\alpha\varepsilon_1} = \frac{\partial C_{\alpha\varepsilon_1}}{\partial P}\, dP + \frac{\partial C_{\alpha\varepsilon_1}}{\partial q}\, dq \tag{6.36}$$

where $\frac{\partial C_{\alpha\varepsilon_1}}{\partial P}$ and $\frac{\partial C_{\alpha\varepsilon_1}}{\partial q}$ are the parameters controlling the contributions to $C_{\alpha\varepsilon_1}$ from q-constant and P-constant tests respectively.

The volumetric creep rate, based on the results of the present investigation and also those of others, can be given as:

$$C_{\alpha v} = A_2 \tag{6.37}$$

where A_2 is a constant for a soil, which may depend to some degree on the stress path.

6.3.1 Evaluation of the Parameters

6.3.1.1 Evaluation of A_2

$C_{\alpha v}$ was found to be independent of η upto a certain value (yield value), after which it starts decreasing. Refer Figs. 5.32(a) and 5.34(a). This is the general picture accepted at the present time (1975). A_2 is then the value of $C_{\alpha v}$ determined from a consolidation test. It is suggested that this value may be obtained by conducting stress path E_1 test from K_o-line. To predict the variation as observed in Figs. 5.32(a) and 5.34(a), it is further suggested that the yield value for $C_{\alpha v}$ may be taken as that obtained for $C_{\alpha\varepsilon_1}$ and this value may be predicted using the Eq. 6.36 as shown in the next section.

6.3.1.2 Evaluation of the Parameters for Eq. 6.36

Axial creep rates ($c_{\alpha\varepsilon_1}$) obtained from E_1 creep test are plotted against ln P. Fig. 6.24(a) shows that $C_{\alpha\varepsilon_1}$ does not depend upon the pressure. E_1 test results indicate that:

$$\frac{\partial C_{\alpha\varepsilon_1}}{\partial P}\, dP = \text{constant} = A_1 \tag{6.38}$$

(A_1 = 0.18 for this clay).

Axial creep rates for B creep test, are plotted in an unconventional manner to account for the stress level dependency

in the formulation. Fig. 6.24(b) shows a linear relationship both for normally & lightly overconsolidated samples. The slope 'a_2' is indicated in the figure for each case.

B creep test indicates:

$$\frac{\partial C_{\alpha\varepsilon_1}}{\partial q} = \frac{a_2}{q_f - q} \tag{6.39}$$

The following relation for creep rate is obtained from Eqs. 6.36, 6.38 and 6.39:

$$dC_{\alpha\varepsilon_1} = A_1 + \frac{a_2}{q_f - q} \quad dq \tag{6.40}$$

This equation clearly predicts the drained creep rates over the whole range of η-values. From Eq. 6.39, it is seen that as $q \to q_f$, $C_{\alpha\varepsilon_1} \to \infty$. This is consistent with the expected trend, as has been observed in this and other investigations. The relationship between $C_{\alpha\varepsilon_1}$ and η so predicted, can be used to obtain the yield value. This is clearly an improvement over Walker's (1969) linear relationship between $C_{\alpha\varepsilon_1}$ and η .

6.3.2. Predictions of Creep Rates for Loading Stress Paths

Table 6.2 and 6.3 show the predictions for the two stress paths. As the creep test results for E stress path are not available, the prediction of $C_{\alpha\varepsilon_1}$ for unloading stress paths

could not be attempted. Tables 6.2 and 6.3 show that the creep rates for various stages along A and G_1 tests are very well predicted. Thus the proposed model has the ability to predict creep rates for various stress paths and stress ratios, at least for the loading stress paths.

A practical method of estimating the creep rates for a footing problem, making use of Lambe's stress path method (originally suggested by Barden, 1969) is given. The axial creep rates $C_{\alpha\varepsilon_1}$ for various stress paths (depending upon the stress increments at different depths calculated from theory of elasticity) can be predicted by the proposed model (using Eq. 6.40). These predicted rates can then be integrated over the thickness of the layer to get the creep settlement rate of the footing (P_{cs}) as:

$$P_{cs} = \int_0^H C_{\alpha\varepsilon_1} \, dz \tag{6.41}$$

where H is the thickness of the layer.

A similar approach is recently suggested by Poulos et. al. (1975), who advocate conducting the relevant triaxial tests, depending upon the stress increments, on samples obtained from different depths. However, Eq. 6.41 can predict the creep rates for any combination of stress increments, once results of B and E_1 creep tests are known.

It is suggested that the prediction of creep rates for retaining structures, slopes etc. could be attempted from a knowledge of creep tests along E, B and B-unloading paths.

The combined approach of the proposed model (i.e., prediction of strains and creep rates) can thus help in solving the stress-deformation-time problems to a reasonably accurate extent. It is, however, felt that the results of exhaustive experimental investigations on different natural clays are needed to ascertain the validity of the concepts presented here.

6.4 EVALUATION OF THE DRAINED CREEP BEHAVIOUR BY RATE PROCESS THEORY*

The recent detailed study of the Berkeley model by Edgers et. al. (1973) has indicated the usefulness of this simple model to predict the undrained creep rates of a field problem. Hardly any effort has been made to verify the predictability of the drained creep rates by this model, as suggested by Singh & Mitchell (1968). In this section, the three parameters needed to predict the strain rates, have been evaluated and the effect of stress path, stress ratio and the stress history on these parameters under drained conditions has been discussed.

The semi-empirical three parameter relationship proposed by Singh and Mitchell is given by:

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* hereafter referred to as Berkeley model.

$$\dot{\varepsilon} = \frac{d\varepsilon}{dt} = A\, e^{\bar{\alpha}\bar{D}} \left(\frac{t_1}{t} \right)^m \qquad (6.42)$$

where,

ε = strain

$\dot{\varepsilon}$ = strain rate

t = time after application of deviator stress

t_1 = reference time

A = $\dot{\varepsilon}$ at $t = t_1$ and $D = 0$

D_{max} = deviator stress causing failure $(\sigma_1 - \sigma_3)$

D = deviator stress, $\sigma_1 - \sigma_3$

$\bar{D}$ = D/D_{max}

m = creep coefficient

$\bar{\alpha}$ = slope of linear portion of $\ln \dot{\varepsilon}$ vs. deviator stress D plot

As for drained tests, $\bar{D} \approx \eta$, η has been made use of in the parameter evaluation.

6.4.1 Evaluation of the Parameters and Discussions

Only the axial strains have been used for this purpose, as the complete time data required for calculating the volumetric and shear strain rates was not available. Figs. 6.25 through 6.28 present the log axial strain rate vs. log time relationships for stress paths A, B, C, G_1 and E_1 for normally consolidated clay

samples. As suggested by Edgers et. al. (1973), the straight lines have been drawn through the experimental points representing the steady state. The relationships are linear for time intervals greater than about 300 minutes. The reason why the experimental points in the beginning donot follow the linear relationship may be due to the effect of the multi-increment tests conducted in this study or may be characteristic of the drained creep behaviour. The Berkeley model has been verified for single-increment tests only (Singh and Mitchell, 1968). The linear relationships as obtained on this plot, confirm the concepts of Singh & Mitchell. These lines are not parallel to each other, thus indicating the slight influence of η on the value of the parameter 'm' (which is the slope of the straight lines on this plot). 'm' varies between 0.4 to 0.8 for different stress paths and stress ratios investigated. The difference in 'm' values for a given value of $\eta (\approx 0.7)$ is about 40 percent for A and C stress-paths, indicating that 'm' may not be constant for a soil. It is likely to be affected by the stress path & stress level. However, Singh & Mitchell (1968) place great emphasis on the creep potential 'm', which they consider to be a material property only slightly influenced by test conditions and the stress level. As 'm' is less than 1 for this clay for different conditions of test, it is indicative of the high creep potential of the clay.

Figs. 6.29 and 6.30 show that for the lightly overconsolidated samples, the pattern of variation is similar to that observed for normally consolidated samples of this clay. The parameter, 'm' , though slightly different for the two stress paths A and B, appears to remain constant with η and the stress history. Hence, with the limited data, it can be concluded that the parameter 'm', does not depend upon the stress level and the stress path. Edgers et. al. (1973) show that this parameter is not constant, but depends upon the test conditions, stress level etc.

Figs. 6.31 through 6.34 represent the log axial strain rate vs. η relationships for varying time intervals. Within the extrapolated range (Figs. 6.25 through 6.30), these are linear lines having a slope of $\bar{\alpha}$ and the reference strain rate 'A' is determined from the intercept of the linear line on the ordinate at a given reference time (t = 10 minutes). Thus 'A' depends upon time (see Fig. 6.35). 'A' value for N.C.C. is larger than the value for OCC , thus indicating a higher order of magnitude of creep rates for normally consolidated samples. This is consistent with the observed behaviour discussed in Chapter 5. 'A' appears to be highly stress path dependent. Its values are 0.002, 0.00024 and 0.000045 for stress paths A, B and C respectively.

$\bar{\alpha}$ varies between 0.75 to 2.15 for normally consolidated clays, and between 3.64 to 5.3 for the lightly overconsolidated clays,

indicating a greater influence of stress level on the rate of creep. This parameter is also influenced by the stress path.

From the foregoing discussion, it is quite clear that the Berkeley model is a useful tool for prediction of steady state drained creep rates, however, parameters A and $\bar{\alpha}$, appear to be highly dependent on Stress history and stress path experienced by the sample. While the proposition of "m" being a soil constant indicative of creep potential appears to be intuitively sound and even tempting, the results of investigations by Edgers et. al. (1973) for undrained creep and the present investigation for drained creep indicate a range of values of "m" for a soil depending on stress history, testing conditions and stress path used in the test. Whether these variation in "m" values for a given soil are mainly due to these effects or simply represent variations within the scatter range arising out of non uniformity of samples and testing errors, need to be thoroughly investigated. At the present time it can thus be concluded that an average "m" value with $\bar{\alpha}$ and A as functions of stress path and stress history could be used in the Berkeley model to predict drained creep rates in the field.

6.5 CONCLUSIONS

A model has been suggested to evaluate stress-strain-time behaviour for K_o-normally consolidated saturated clays. Tests

needed and the procedures to evaluate the relevant parameters have been outlined. It has been shown that for all loading stress paths, the predictions are in very good agreement with the experimental data. In case of unloading stress paths, the predictions are not as good and the discrepencies are likely to be due to lack of appropriate and complete test data. However, the model is able to, at least, give the proper trends.

It has been suggested that the model can give reasonable predictions of K_0 value and behaviour during undrained shear. The values of σ_3'/σ_1' during K_0-unloading can also be estimated. The flow rule ($\frac{\dot{\varepsilon}}{v}$) as obtained here is almost identical to the findings of Roscoe and Burland (1968) and Newland (1973). The suggested semi-empirical approach appears to be consistent with the current emphasis by other research workers in this area at the present time (see Wroth 1968 and Lewin 1975). Such an approach is shown to be highly versatile and can be extended to the understanding of the stress-strain-time behaviour under more general stress conditions.

It has been suggested that the proposed method can be used to calculate settlement of footings on soft clays by adopting the Lambe's stress path approach. The same is shown to be applicable for drained creep rates.

The Berkeley model is shown to be able to predict the steady state drained creep rates, if an average 'm' value and $\bar{\alpha}$ and 'A' as functions of stress path and stress history are used in the relationship.

Table 6.1

Suggested Basic Tests to Evaluate the Relevant Parameters in the Proposed Model for Different Stress Paths

S.No.	Stress paths to be predicted by the model.		Recommended basic tests needed for parameter evaluation
	Stress path*	designation of stress paths used	
1	Δp and Δq increasing [$243° \leq \theta \leq 360°$]	A_1, G_1, F_1	E_1 and B-loading
2	Δp decreasing and Δq increasing [$135° \leq \theta \leq 243°$]	C, D	E and B-loading
3	Δp and Δq decreasing [$63° \leq \theta \leq 135°$]	F, G	E and B-unloading
4	Δp increasing and Δq decreasing [$0 \leq \theta \leq 63°$]	-	E_1 and B-unloading

*Refer Figs. 3.3(a) & (b) for details.

Table 6.2

PREDICTION OF CREEP RATES
STRESS PATH 'A'
(NCC)

n	$c_{\alpha\varepsilon_1}$	
	PREDICTED	Measured
0.468	0.297	-
0.534	0.61	-
0.595	0.93	1.05
0.656	1.28	1.23

Table 6.3

PREDICTION OF CREEP RATES
STRESS PATH 'G_1'
(NCC)

n	$c_{\alpha\varepsilon_1}$	
	Predicted	Measured
0.415	0.243	0.425
0.430	0.485	0.425
0.443	0.725	0.543
0.455	0.96	0.62

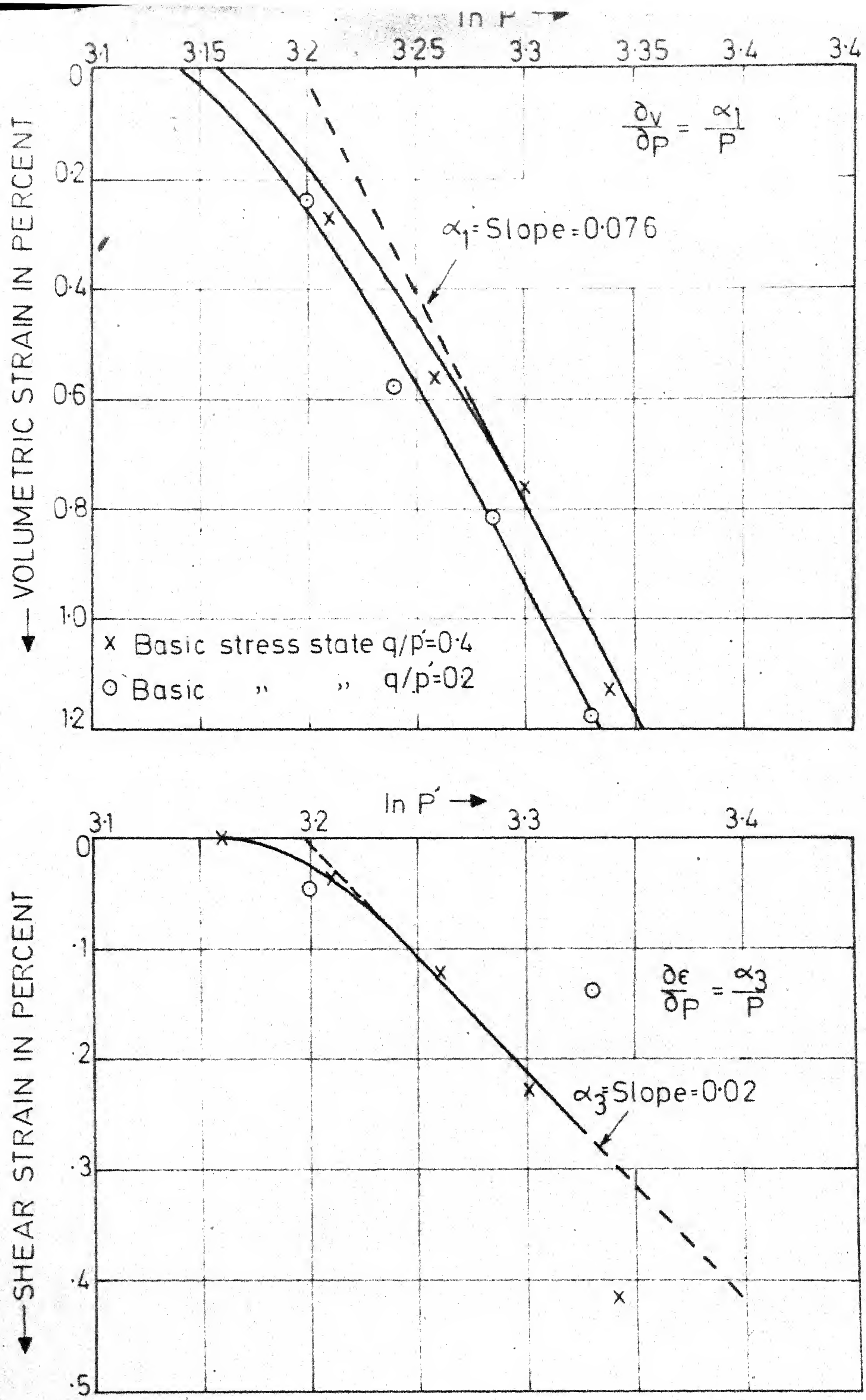


FIG. 6·1 VARIATION OF STRAIN WITH ln P′ FOR 'E$_1$' STRESS PATH

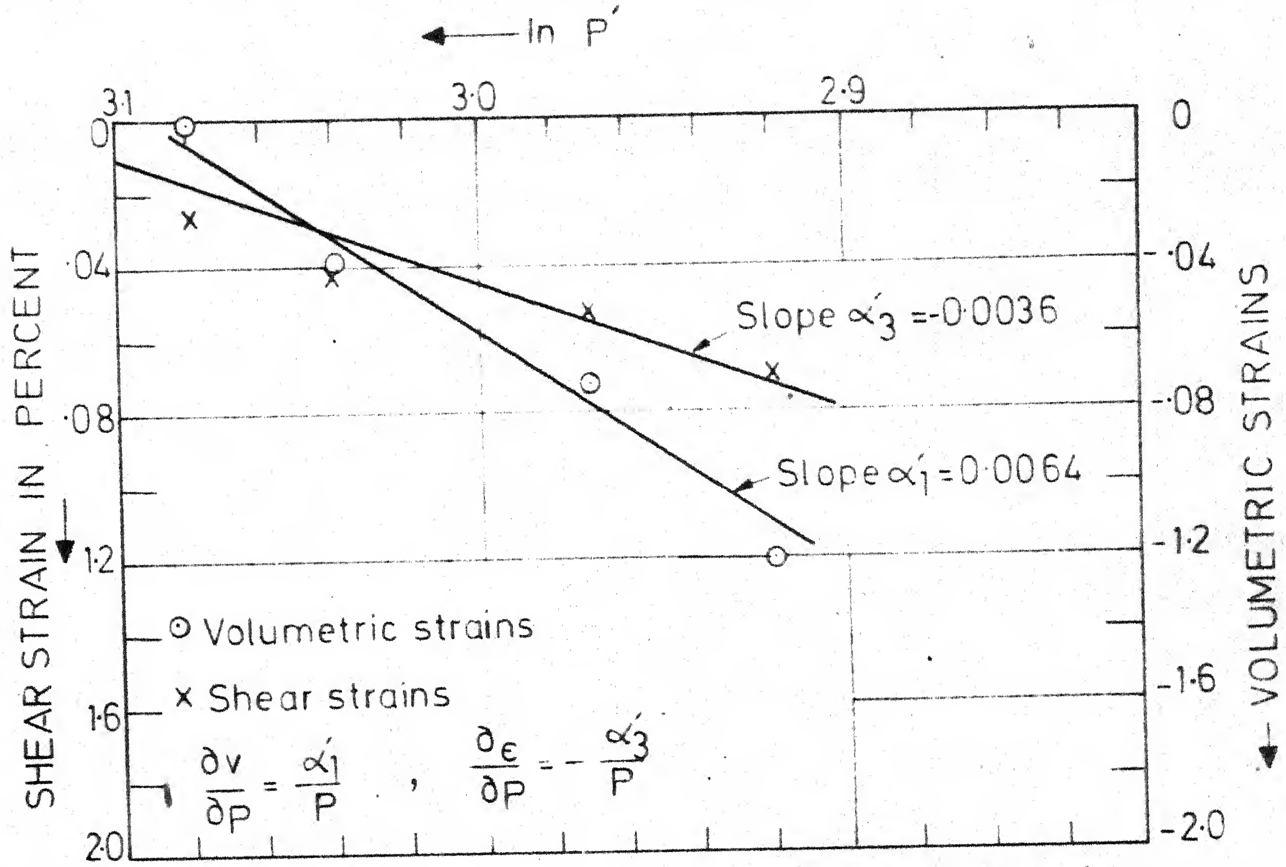


FIG.6.2 VARIATION OF STRAIN WITH ln P′ FOR 'E' STRESS PATH (Basic stress state q/p′=0.2)

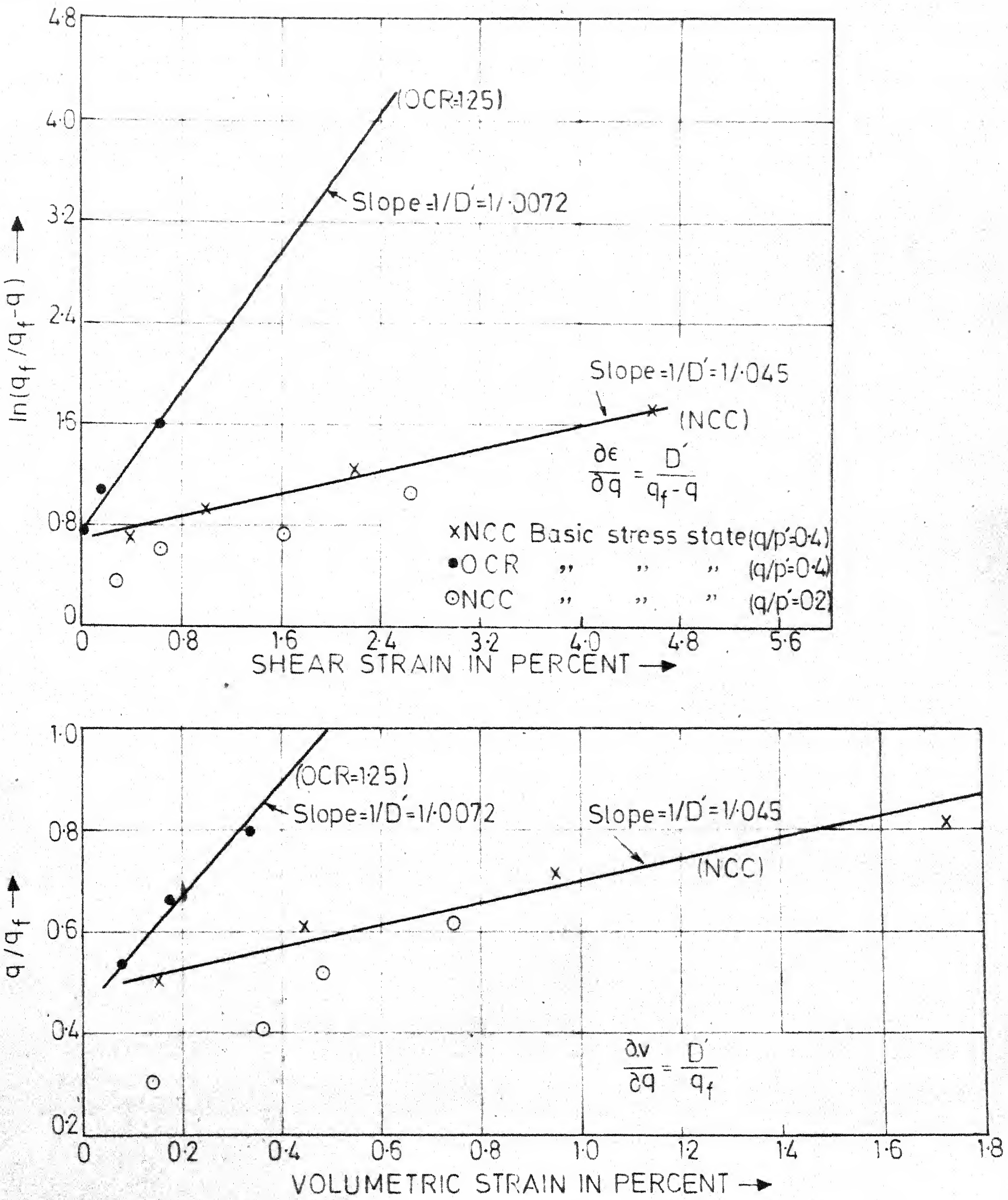


FIG.6.3 P-CONSTANT (STRESS PATH 'B') TEST ON NORMALLY & LIGHTLY OVER CONSOLIDATED RANN OF KUTCH CLAY

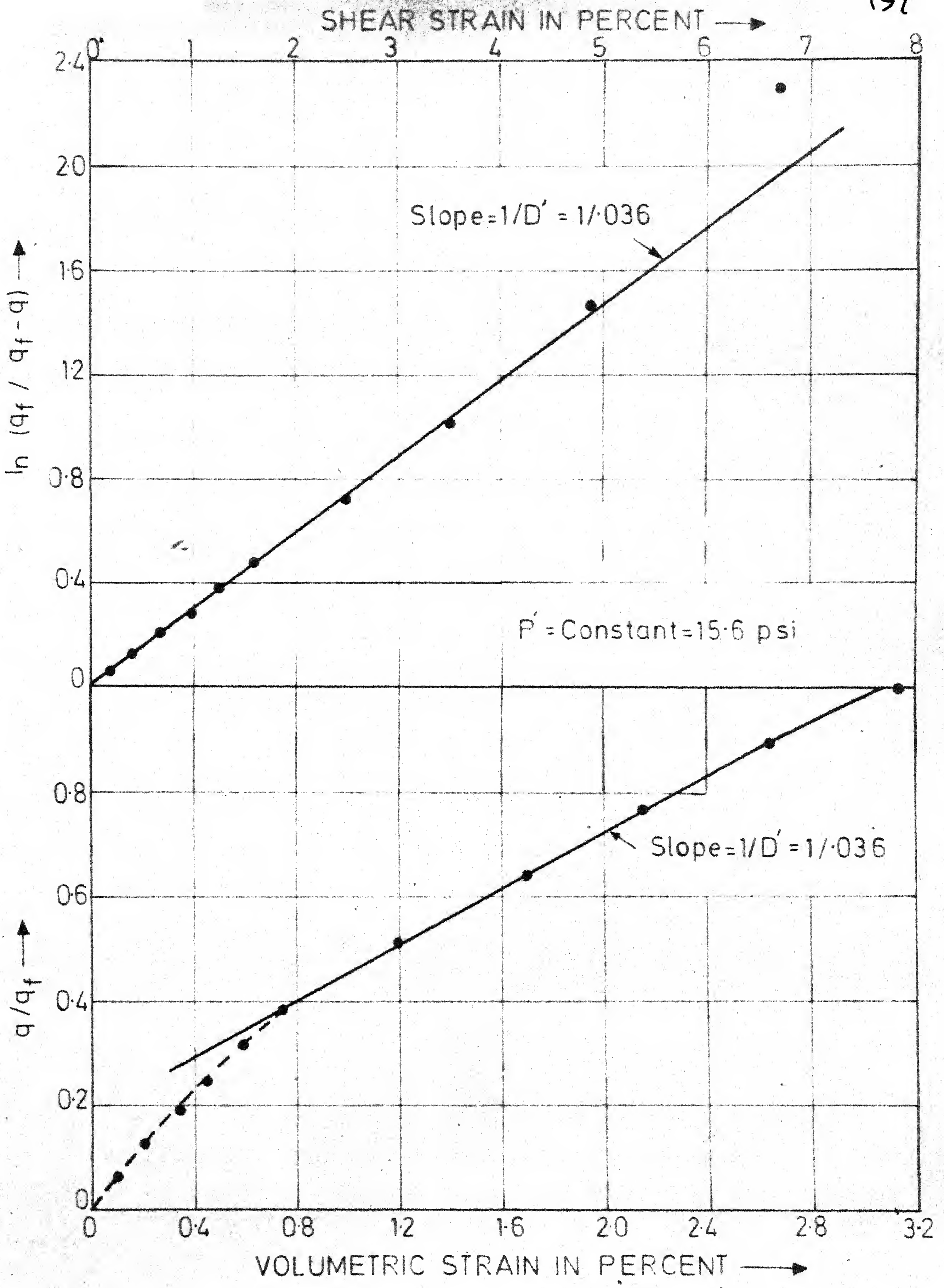


FIG. 6·4 STRESS PATH 'B' TEST BY VARADARAJAN (1973) ON ISOTROPICALLY CONSOLIDATED CLAY

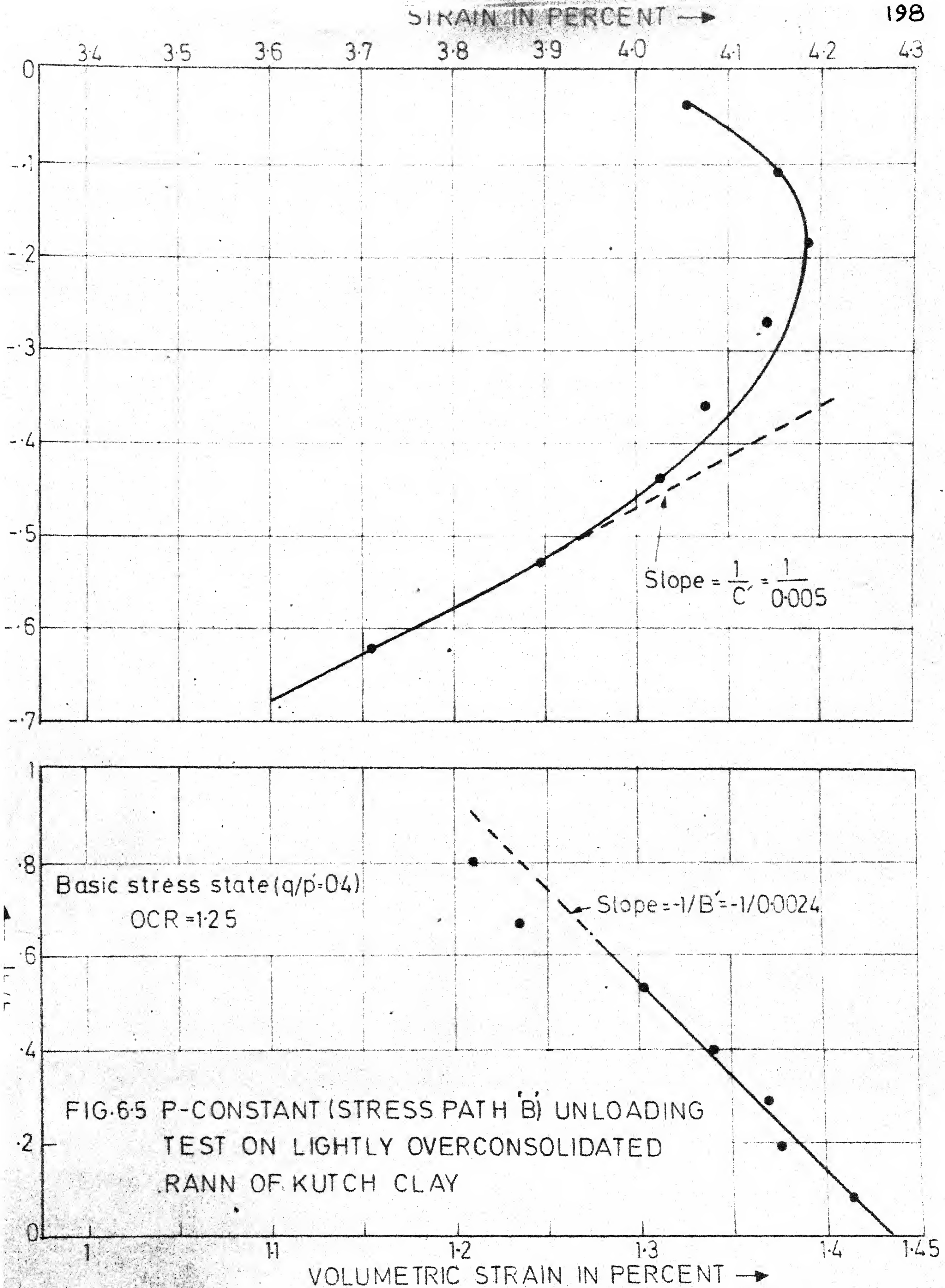


FIG. 6·5 P-CONSTANT (STRESS PATH 'B') UNLOADING TEST ON LIGHTLY OVERCONSOLIDATED RANN OF KUTCH CLAY

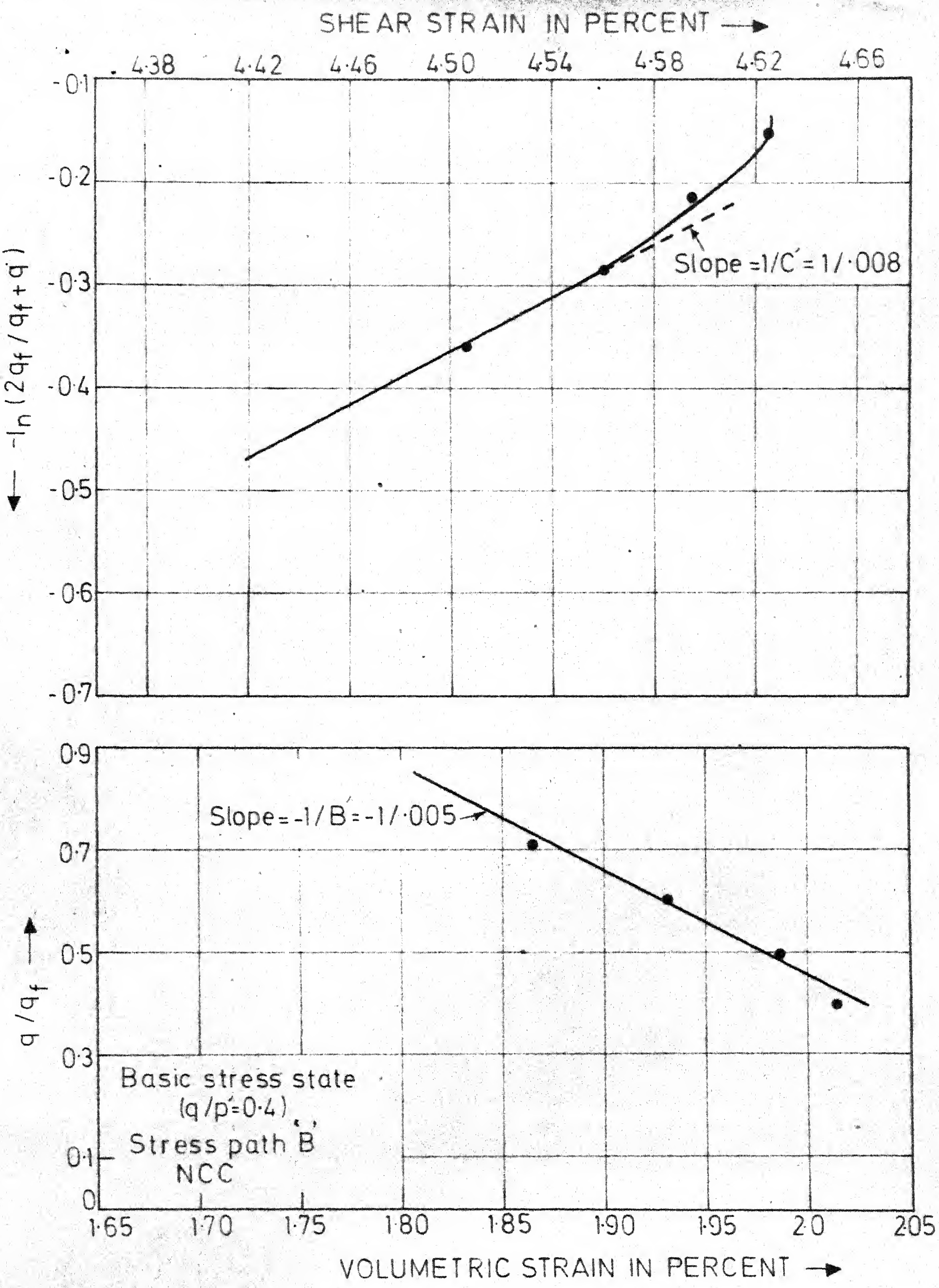


FIG.6·6 P-CONSTANT (STRESS PATH 'B') UNLOADING TEST ON NORM--ALLY CONSOLIDATED RANN OF KUTCH CLAY

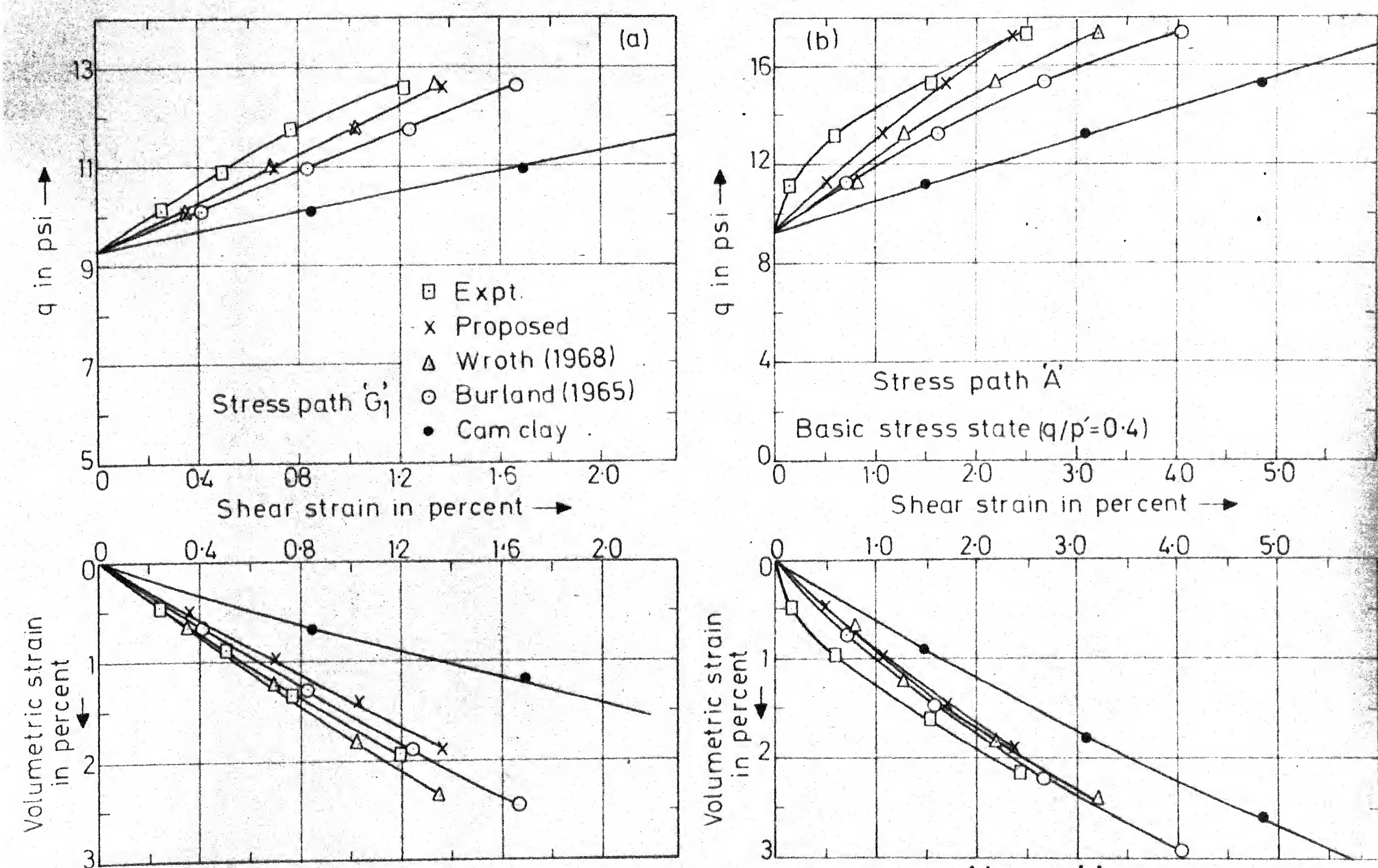


FIG. 6.7 COMPARISON OF THE PREDICTION OF STRESS PATH 'A' AND 'G'1' TEST RESULTS BY VARIOUS MODELS

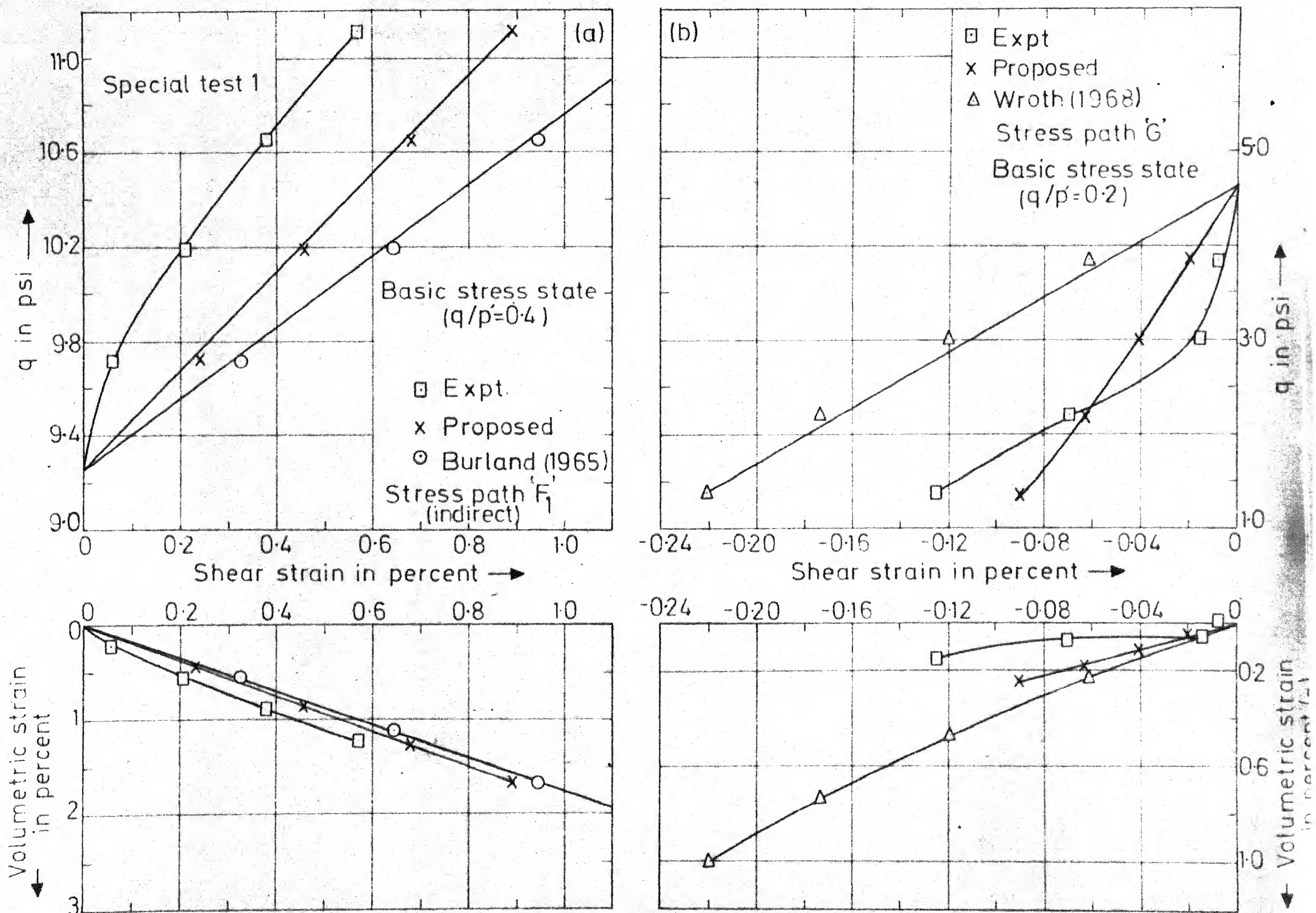


FIG. 6·8 COMPARISON OF THE PREDICTIONS OF THE TEST RESULTS BY VARIOUS MODELS

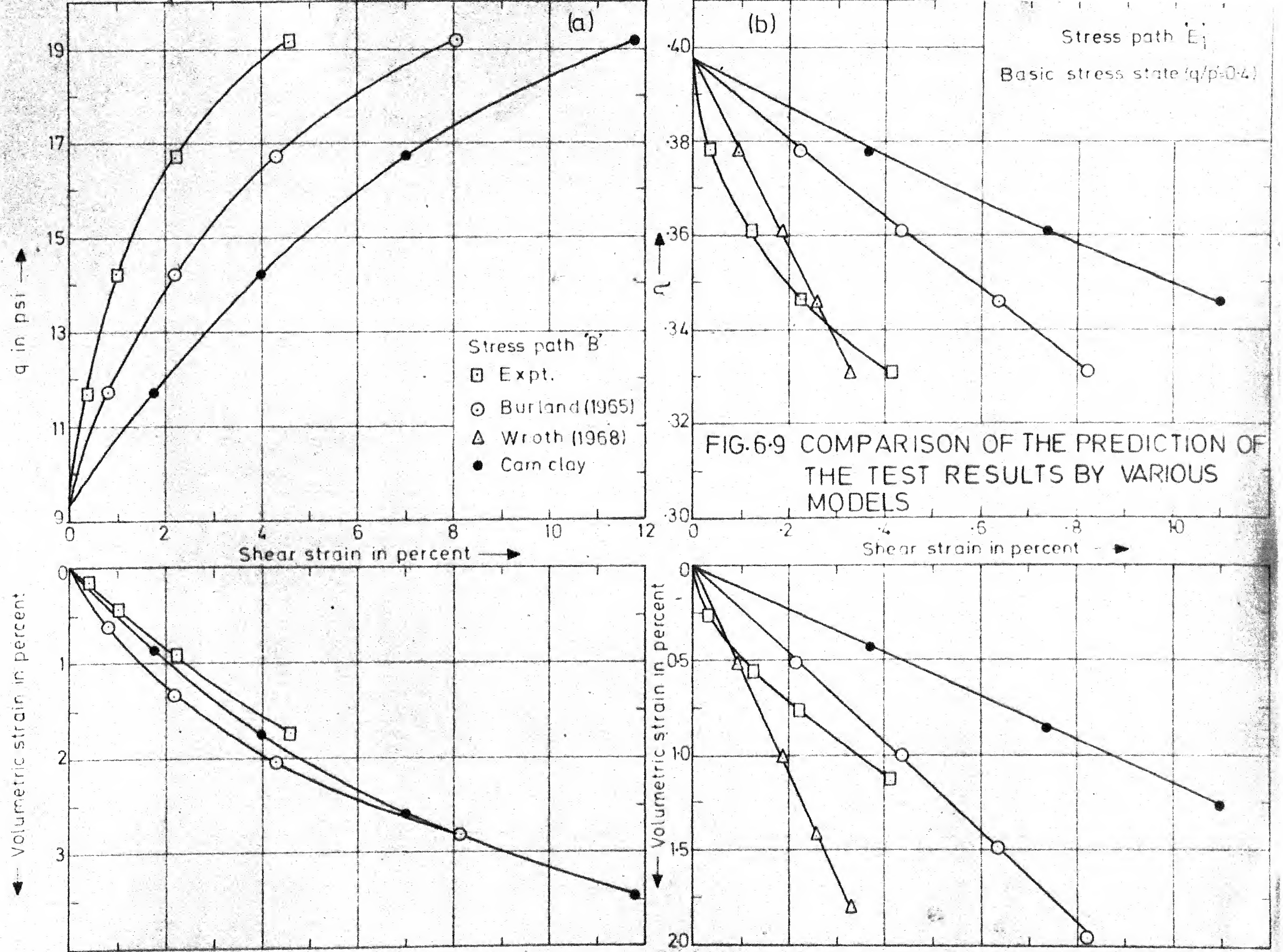


FIG. 6·9 COMPARISON OF THE PREDICTION OF THE TEST RESULTS BY VARIOUS MODELS

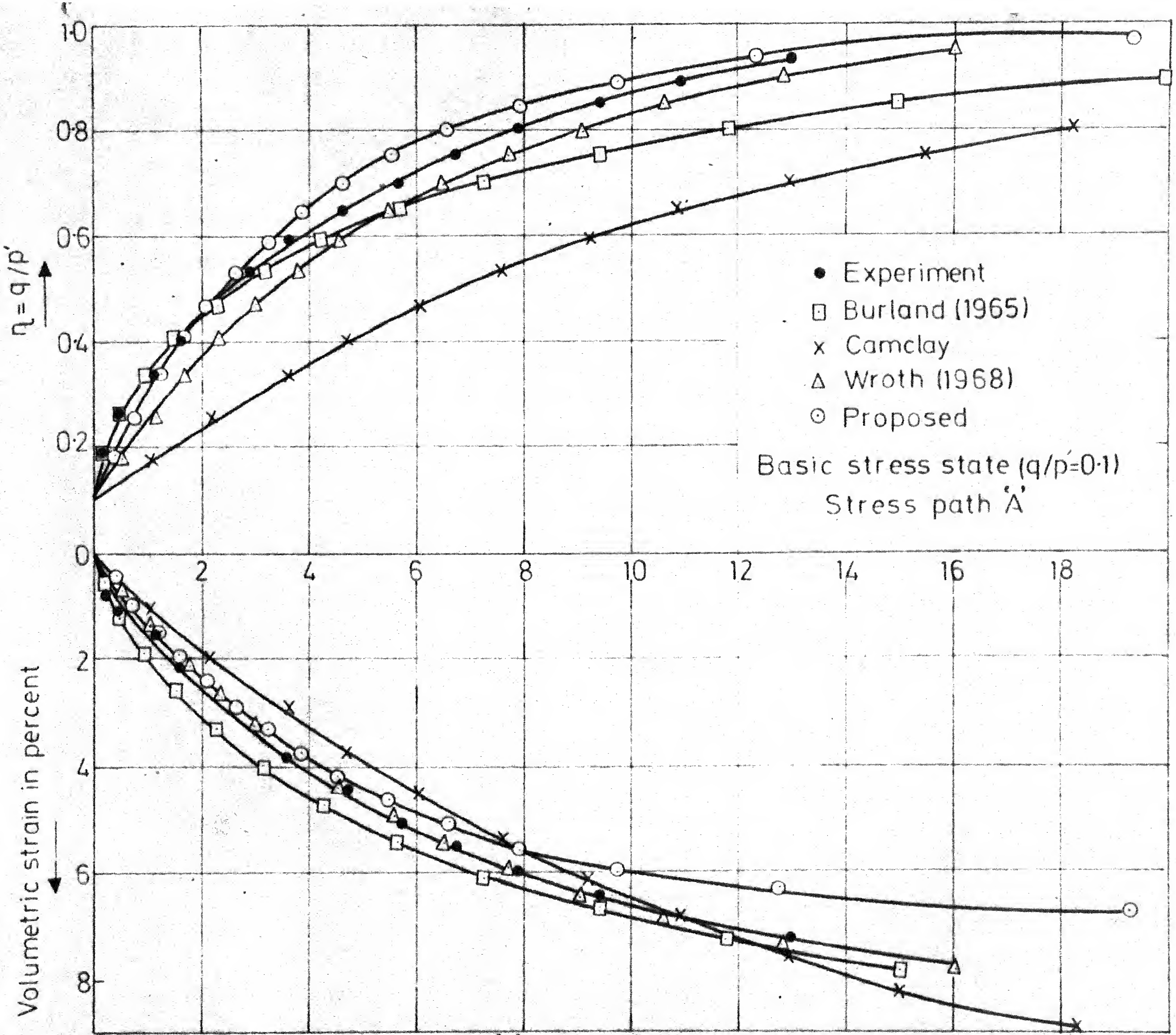
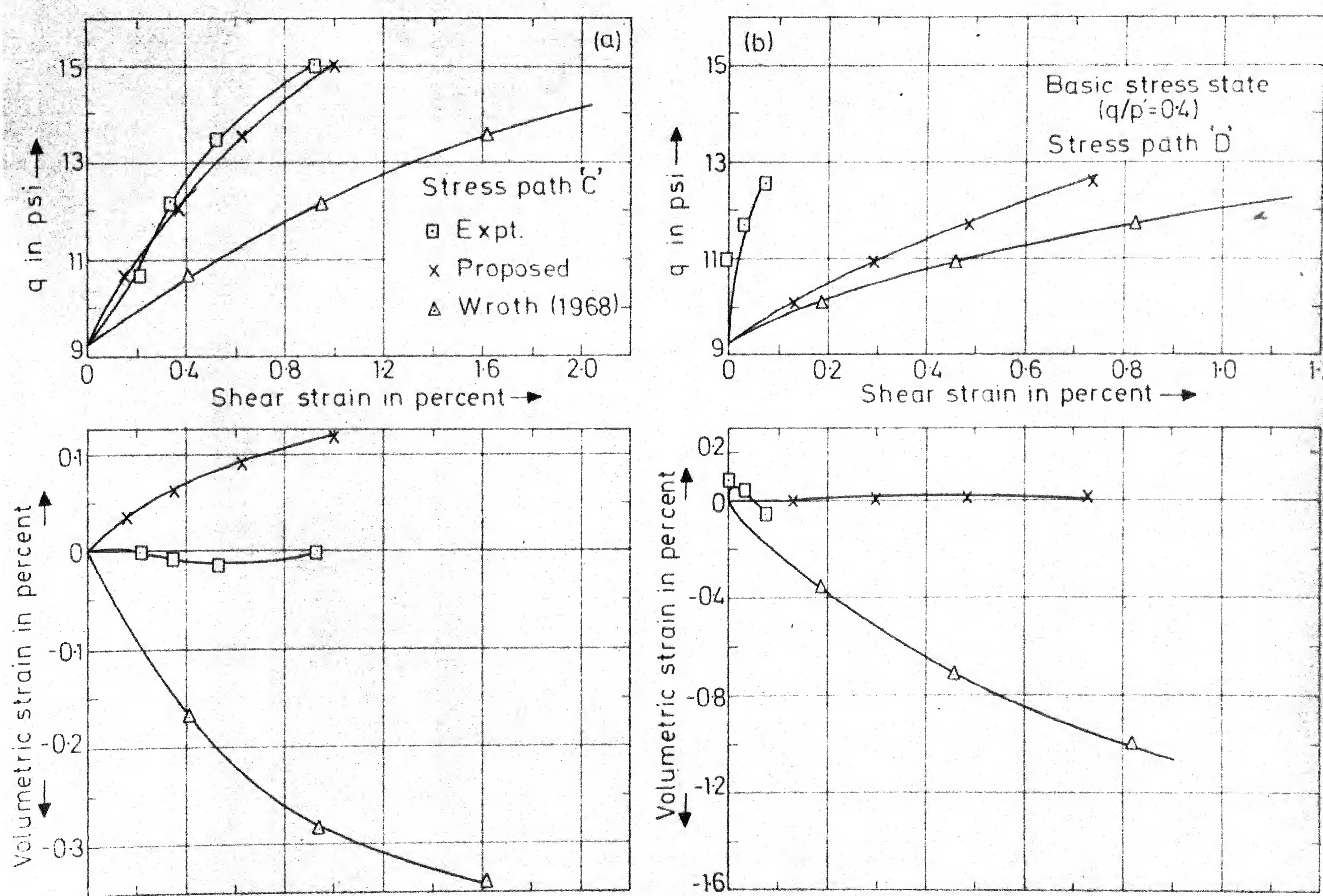


FIG. 6·10 PREDICTION OF STRESS PATH 'A' TEST BY VARIOUS MODELS (M=1·0, λ=0·3, K=0·1)



FIG·6·11 COMPARISON OF THE PREDICTION OF THE TEST RESULTS BY VARIOUS MODELS

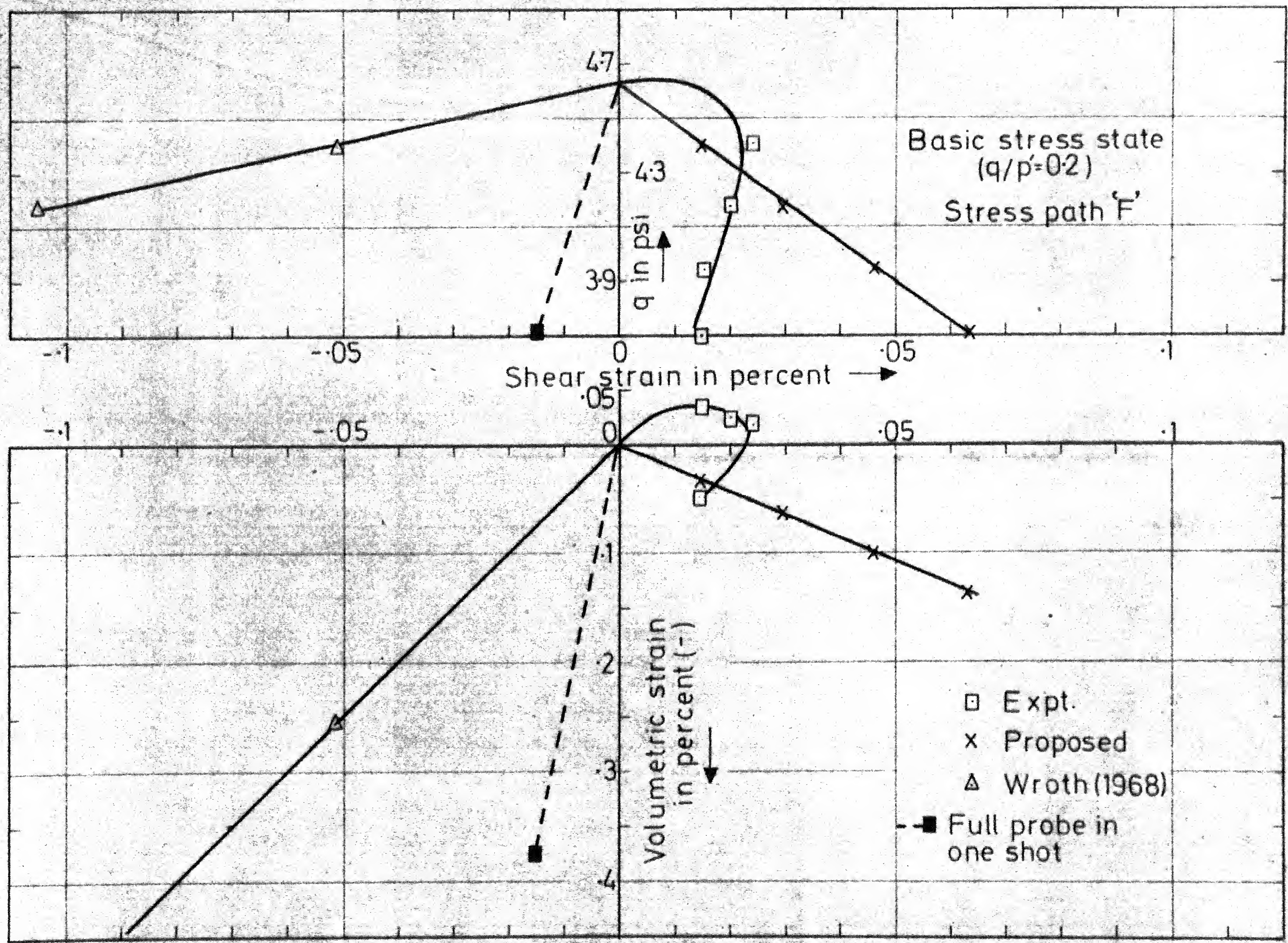


FIG. 6·12 COMPARISON OF THE PREDICION OF THE TEST RESULTS BY VARIOUS MODELS

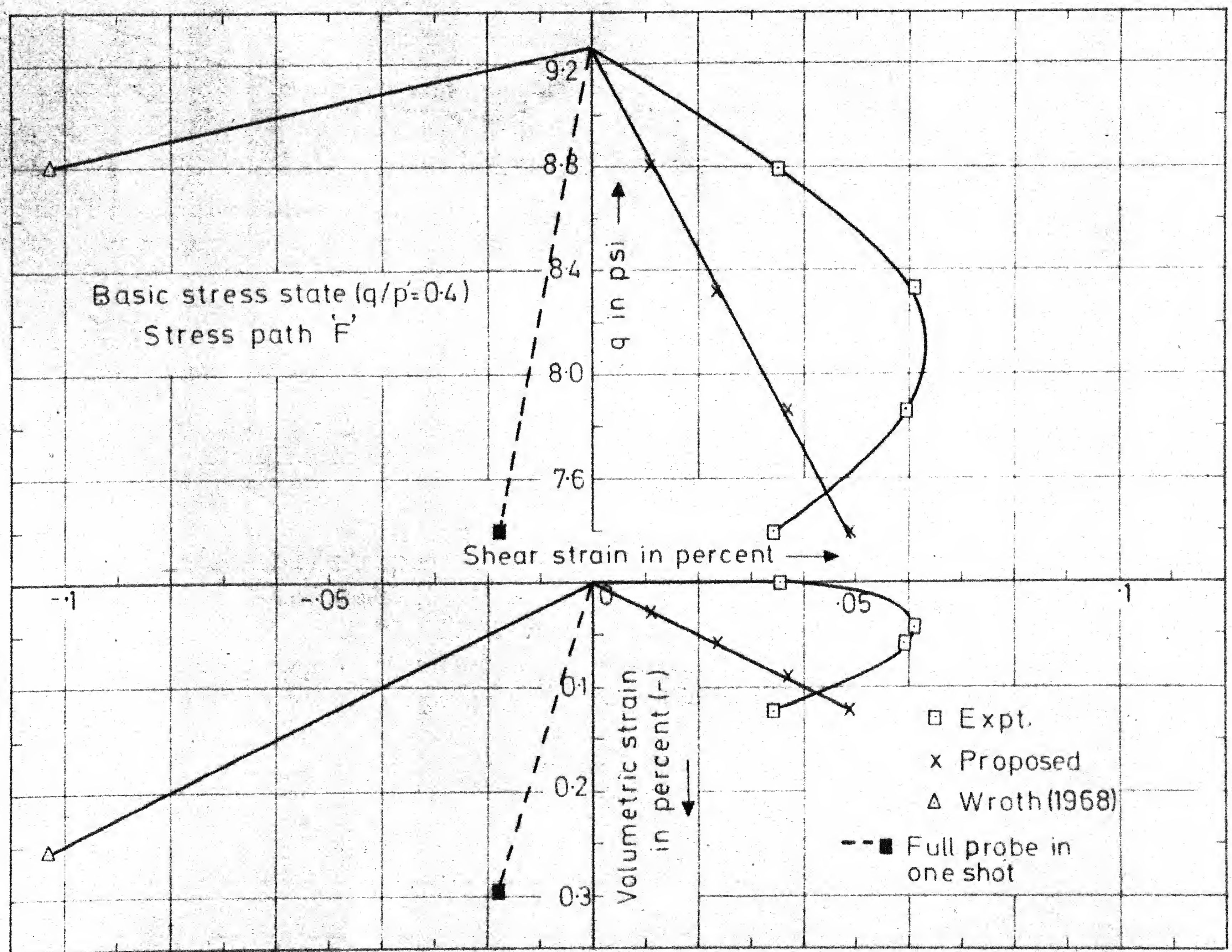


FIG.6.13 COMPARISON OF THE PREDICTION OF THE TEST RESULTS BY VARIOUS MODELS

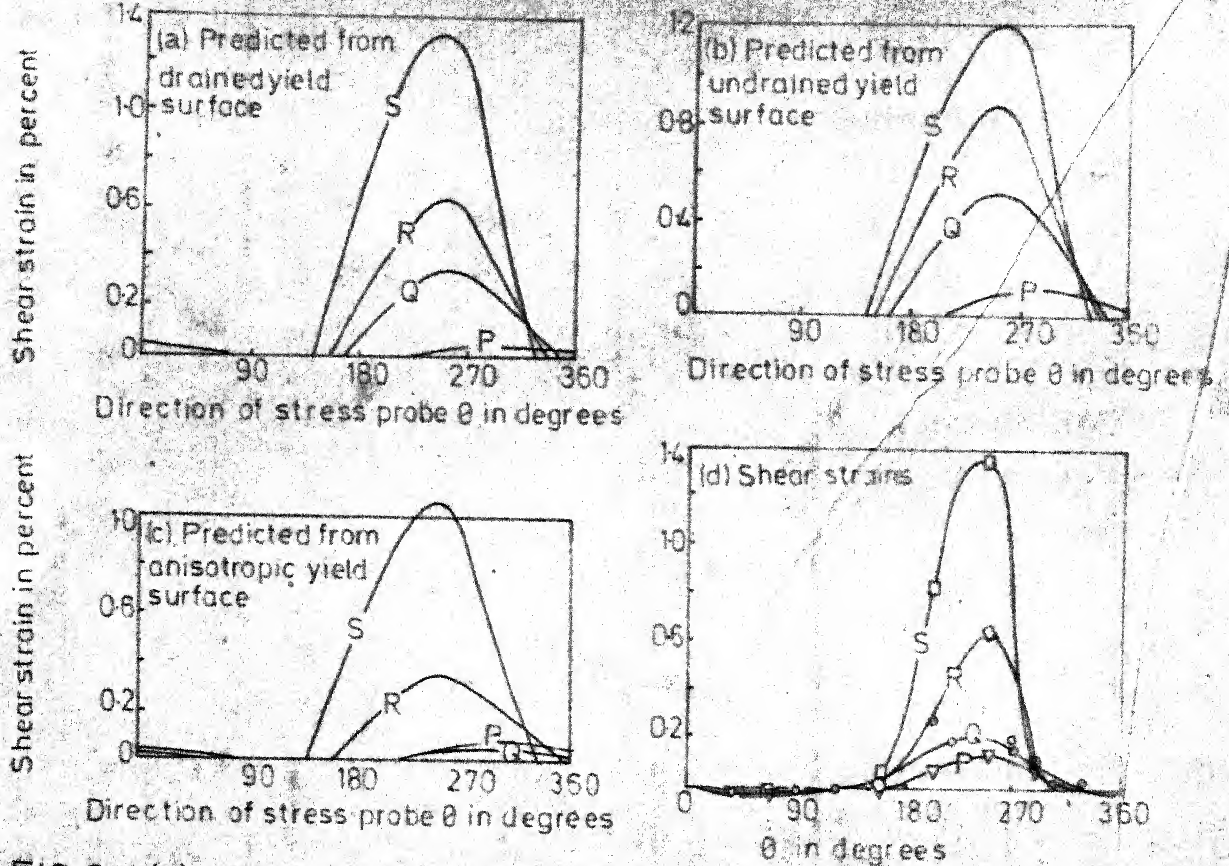


FIG 6.14 (a) PREDICTION OF SHEAR STRAINS (After Lewin & Burland, 1970)

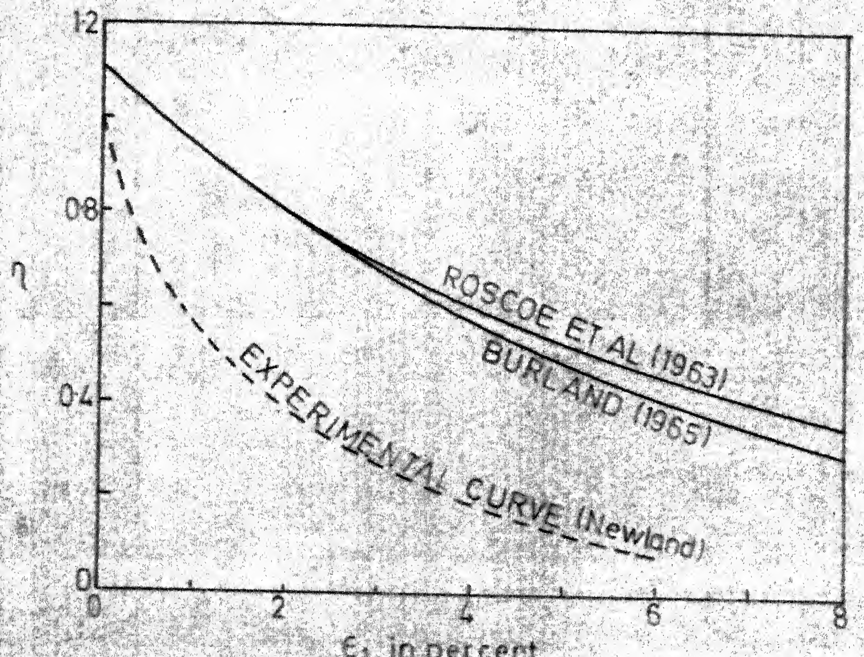


FIG 6.14(b) PREDICTION OF AXIAL STRAIN FOR 'E1' STRESS PATH (After Newland, 1973)

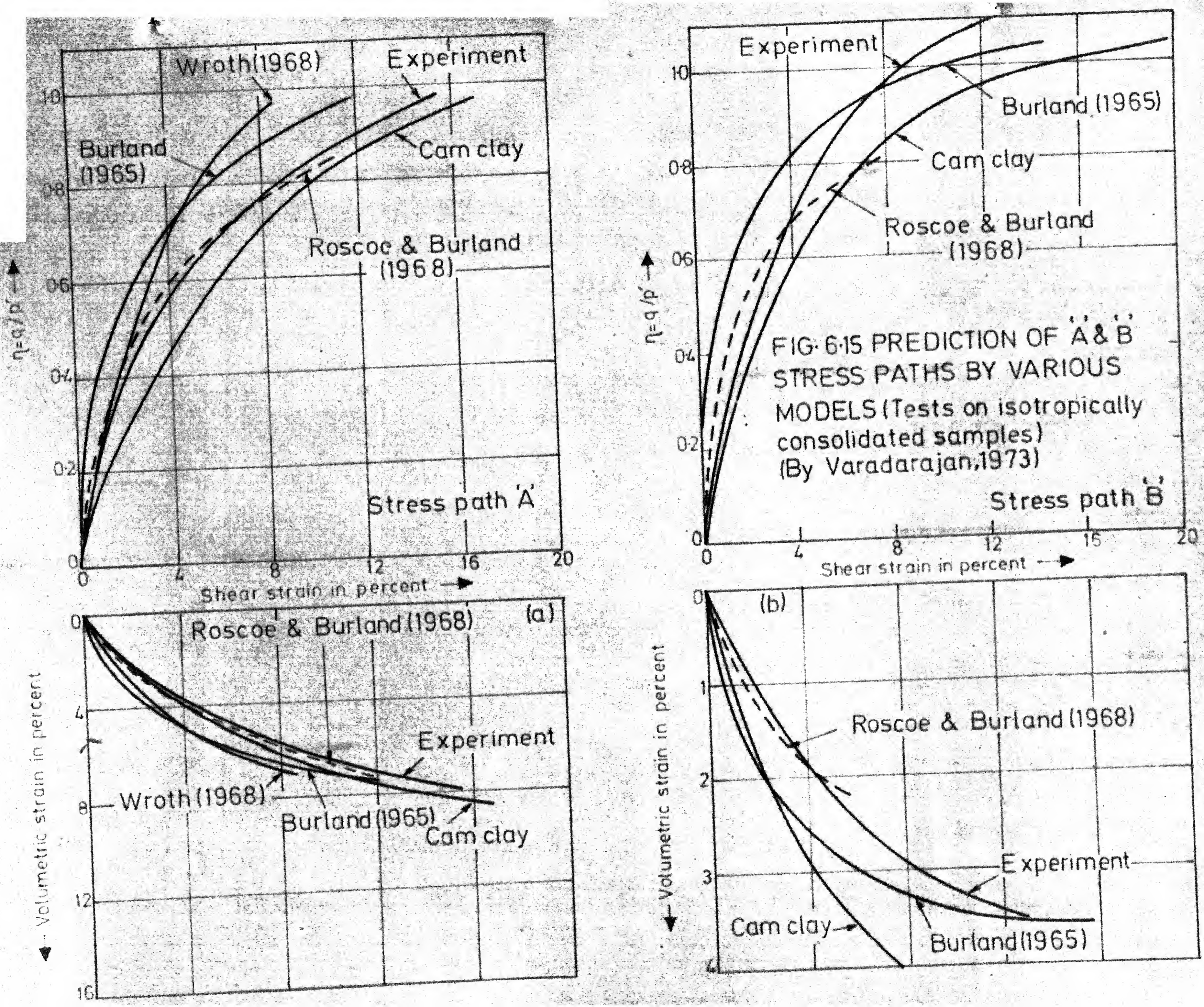


FIG. 6.15 PREDICTION OF 'A' & 'B' STRESS PATHS BY VARIOUS MODELS (Tests on isotropically consolidated samples) (By Varadarajan, 1973)

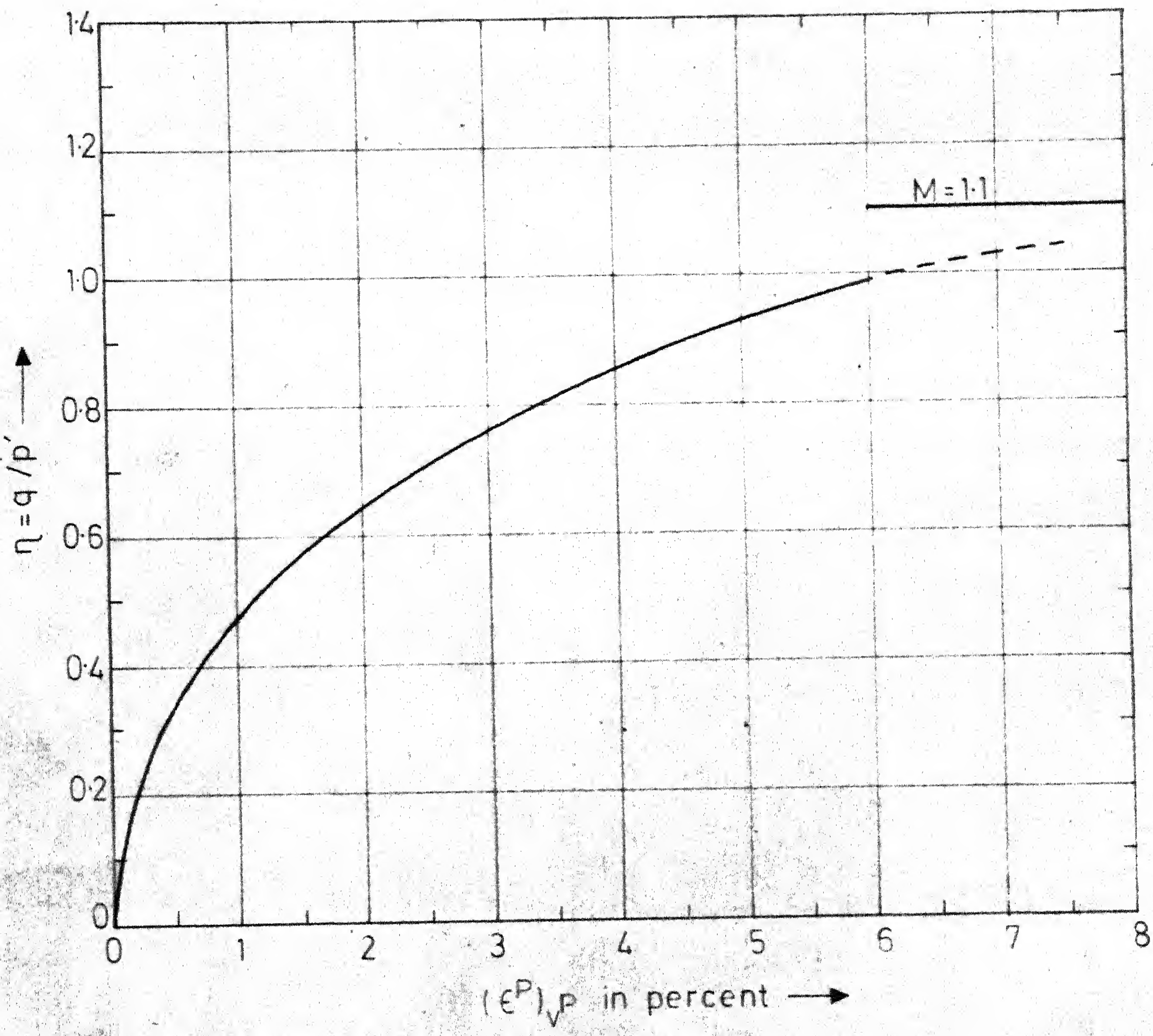


FIG.6·16 η-$(\epsilon^P)_V P$ RELATIONSHIP FOR RANN OF KUTCH FROM TESTS BY RAHMAN (1972)

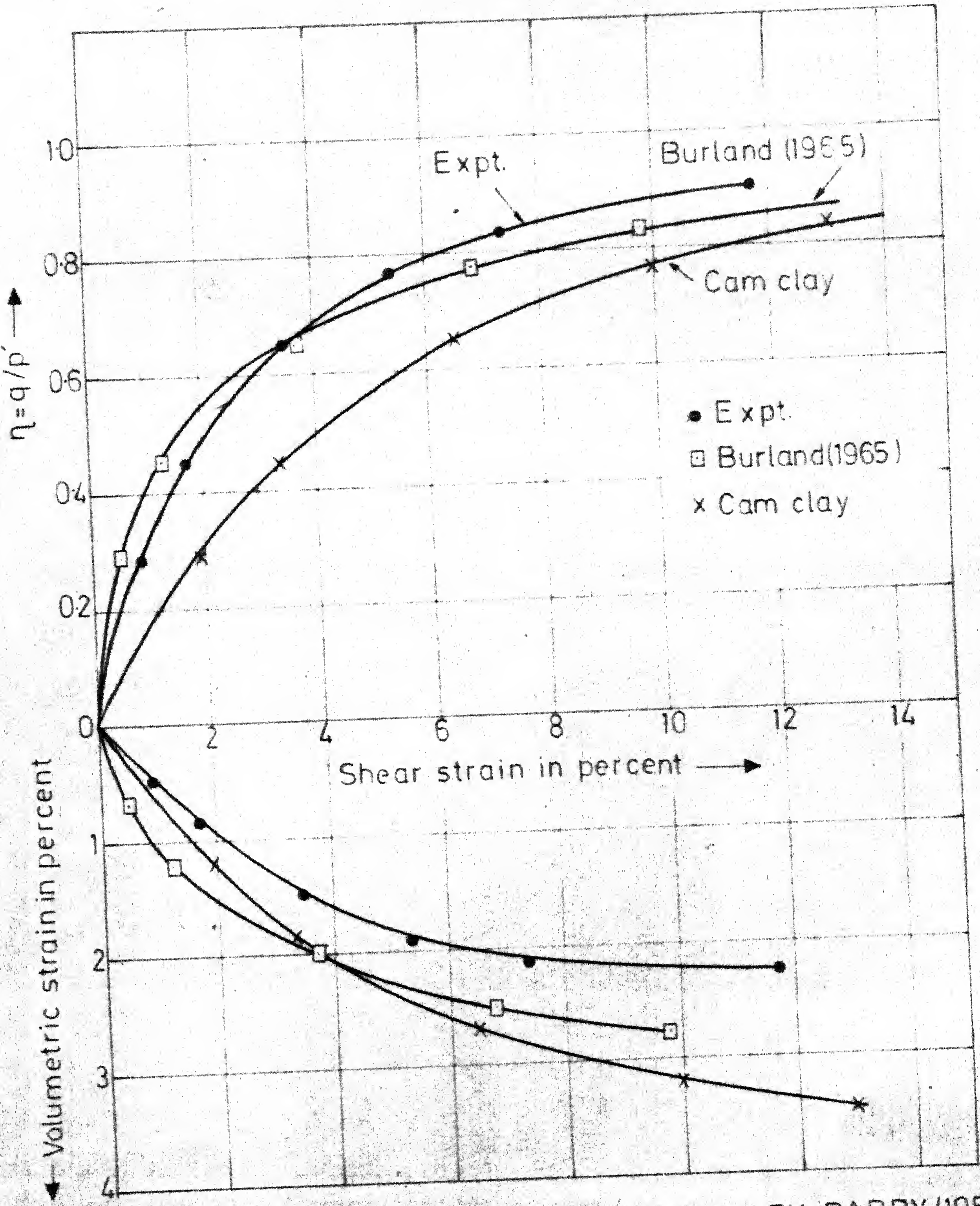
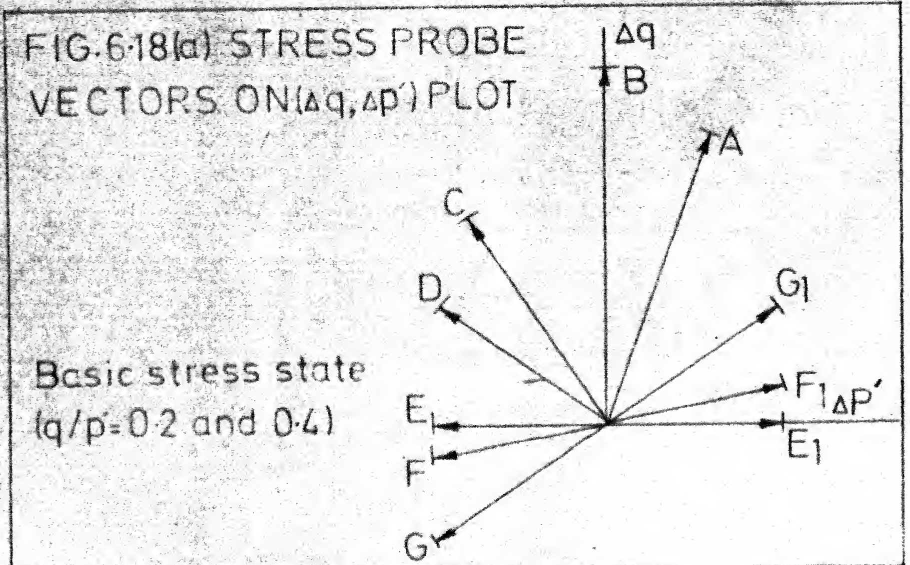
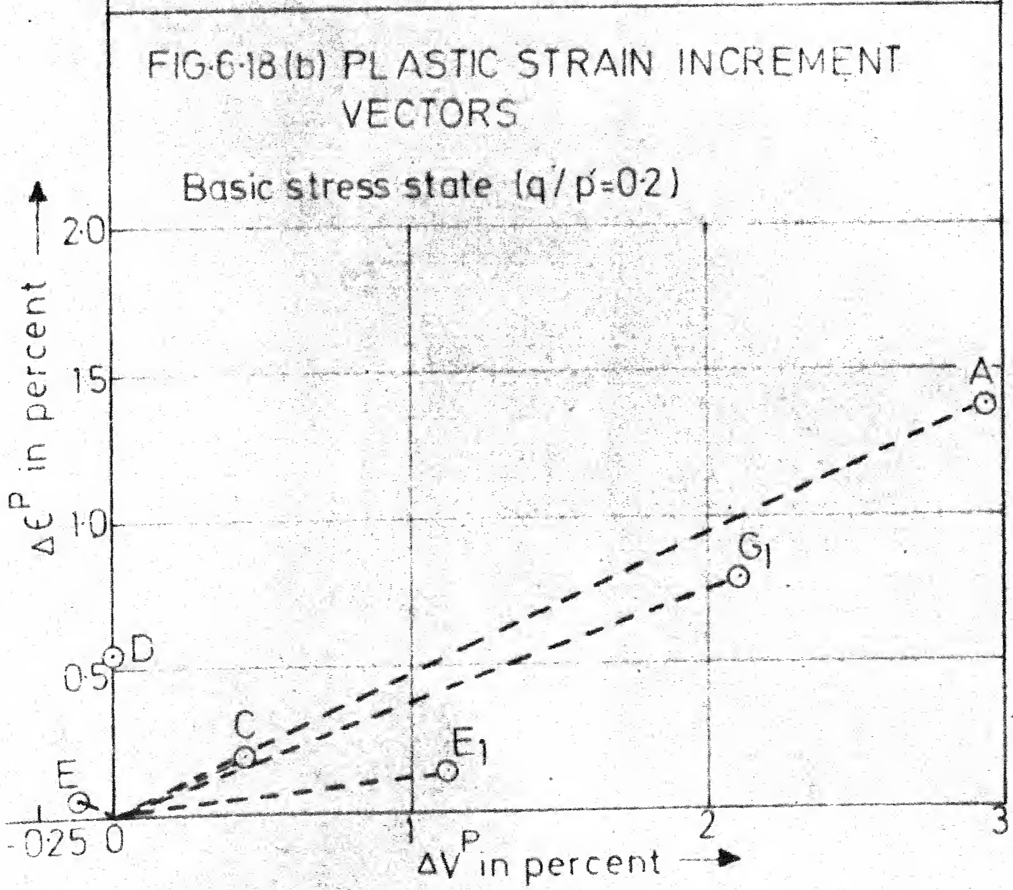


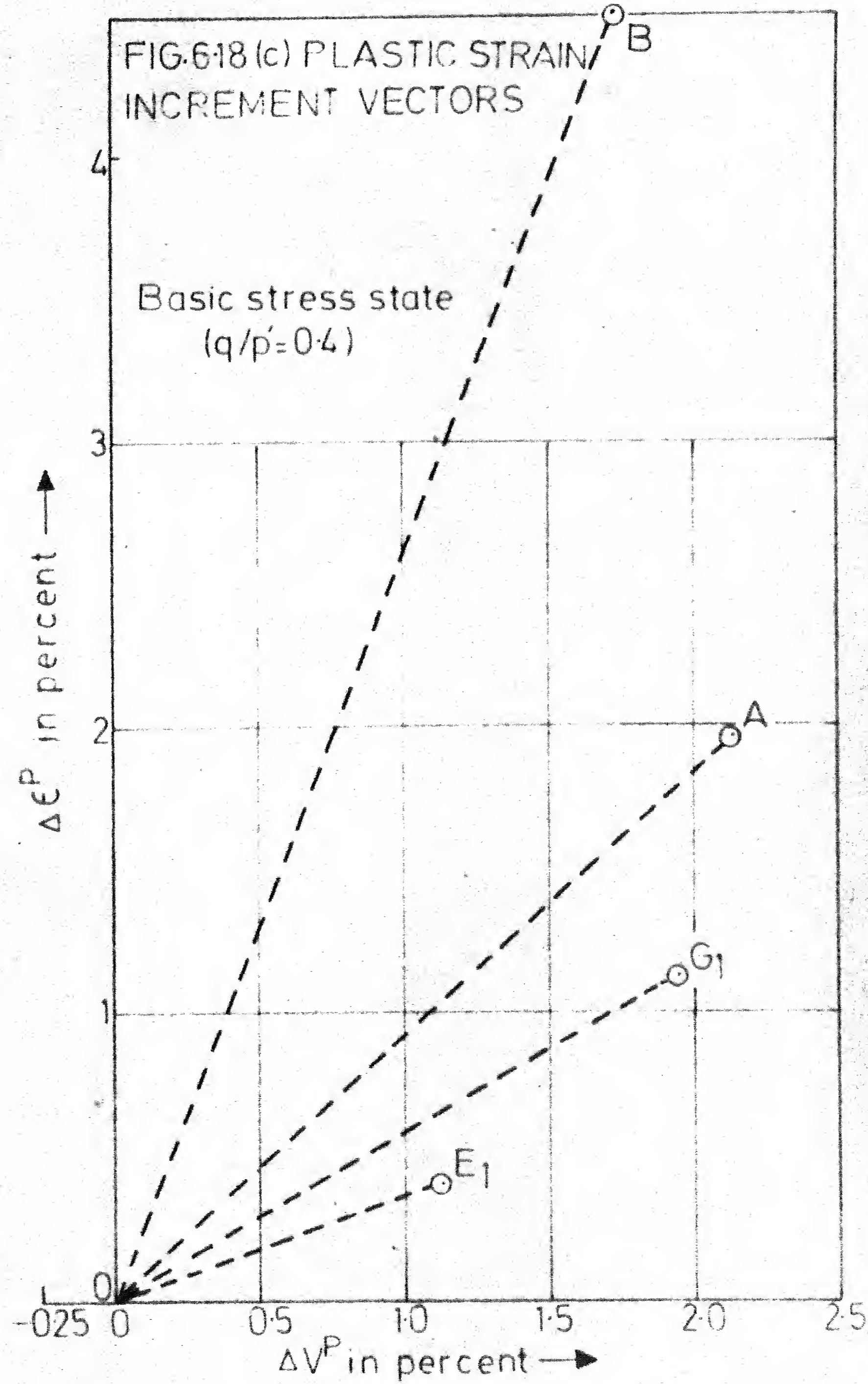
FIG. 6·17 P-CONSTANT TEST ON WEALD CLAY BY PARRY (1956) (P = 30 psi) (on isotropically consolidated sample)



FIG·6·18(a) STRESS PROBE VECTORS ON(Δq,Δp′) PLOT



FIG·6·18(b) PLASTIC STRAIN INCREMENT VECTORS



FIG·6·18(c) PLASTIC STRAIN INCREMENT VECTORS

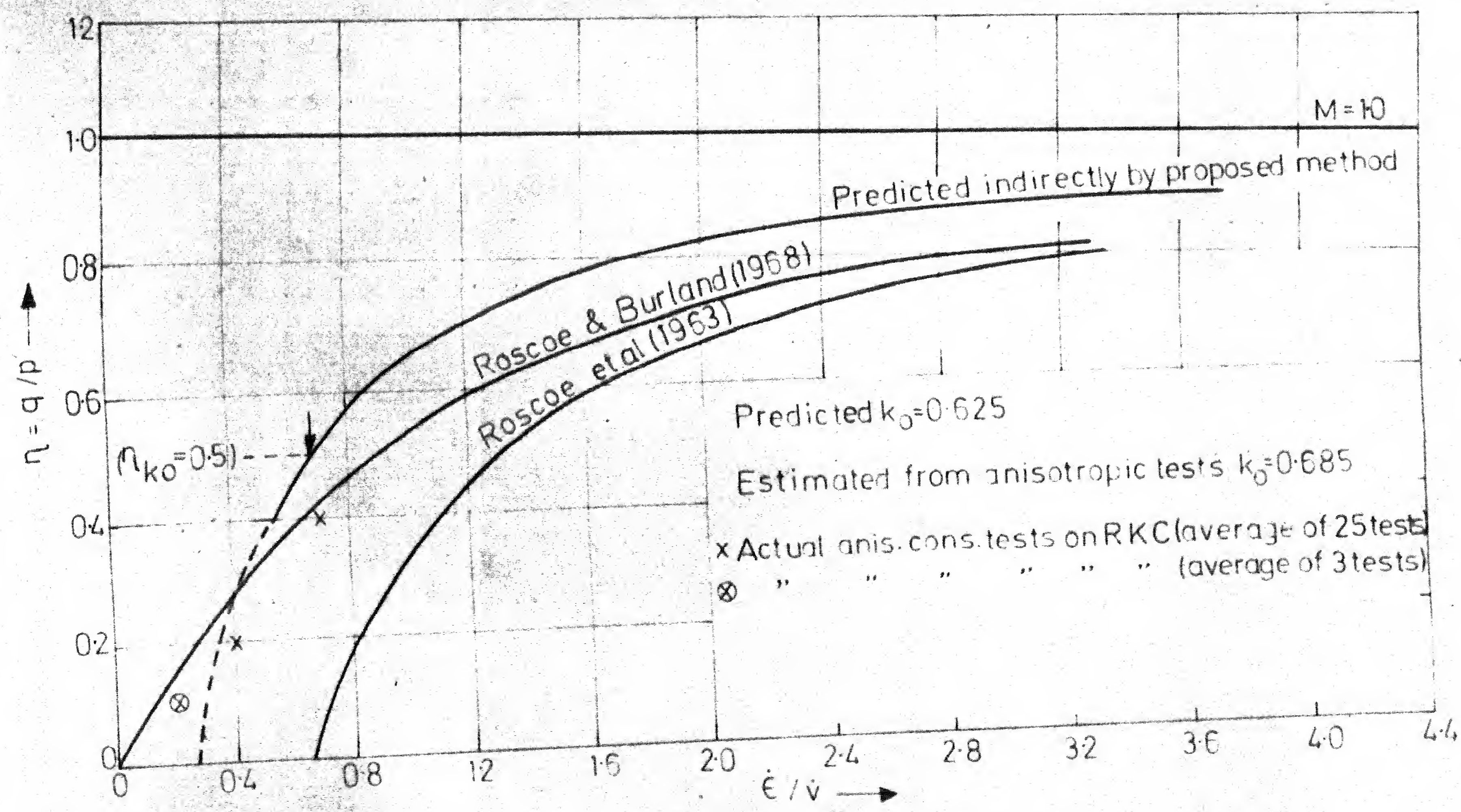


FIG. 6.19 ANISOTROPIC CONSOLIDATION OF RANN OF KUTCH CLAY

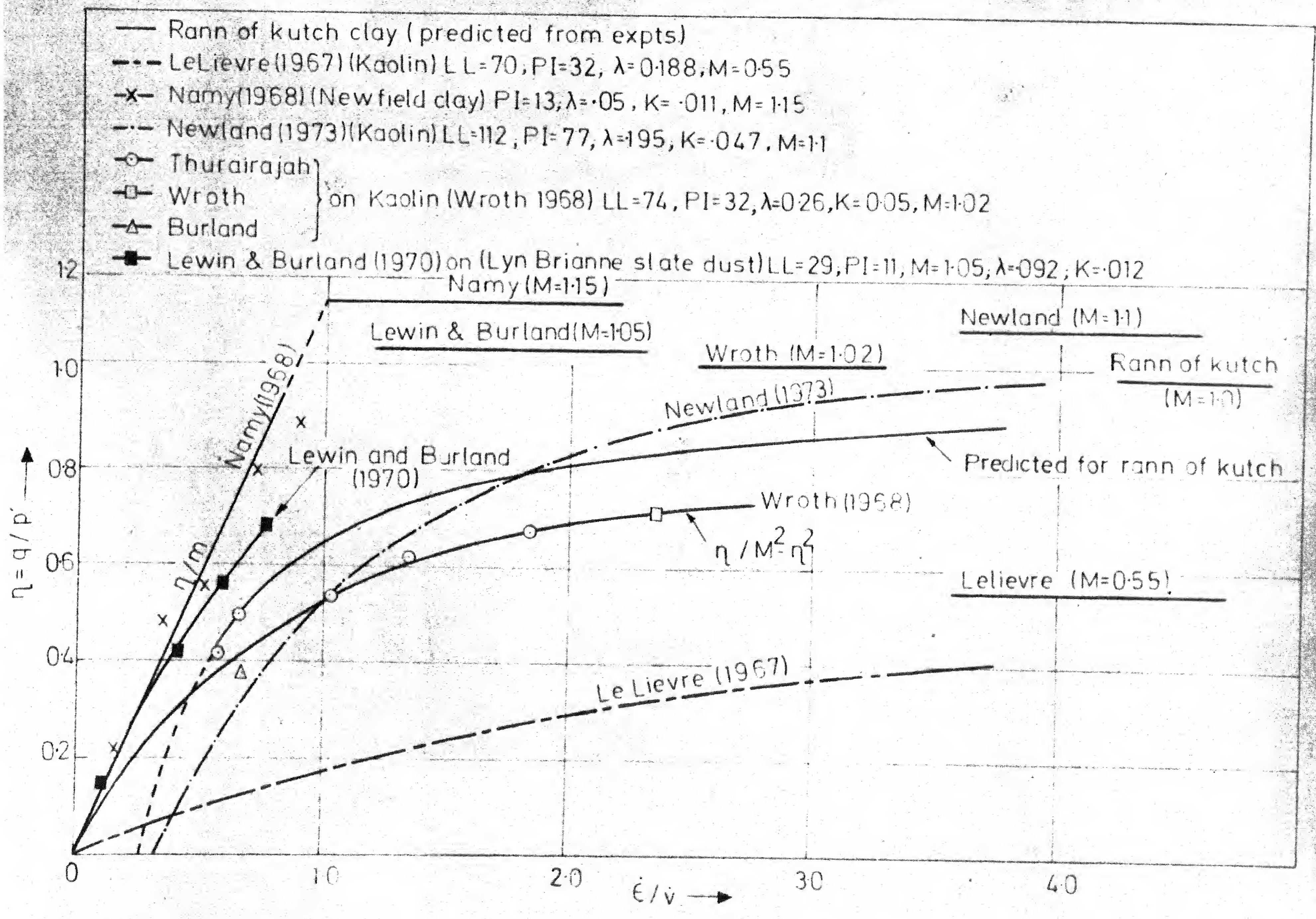
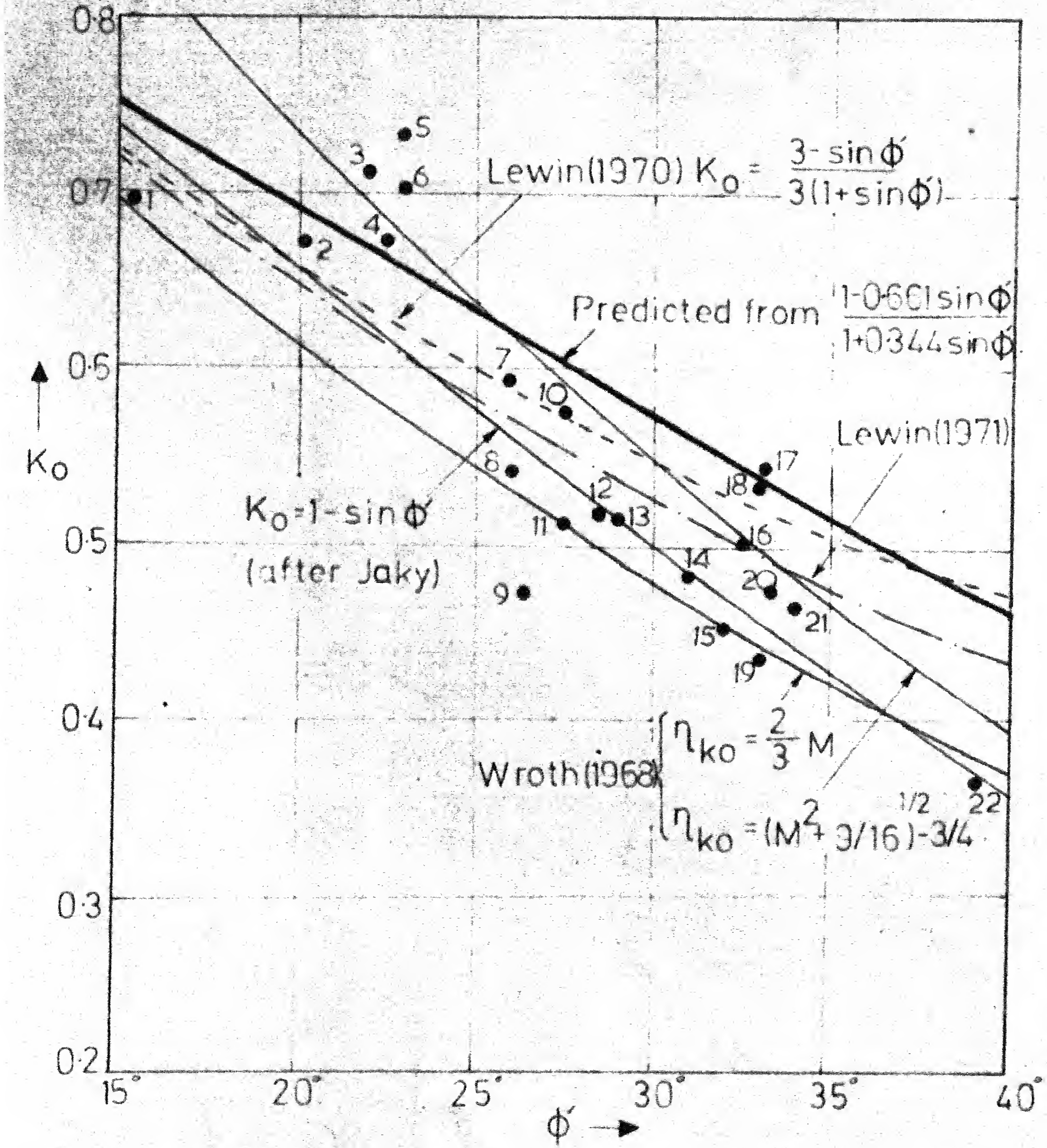


FIG. 6·20 ANISOTROPIC CONSOLIDATION TESTS ON OTHER SOILS



Legend

No	Soil	Author
1	Bearpaw shale	Brooker (1964)
2	Kaolin	Bishop (1958)
3	Weald clay	Bishop (1958)
4	London clay	Brooker (1964)
5	Gault clay	Thompson(1962)
6	Kaolin	Burland (1967)
7	Weald clay	Sowa (1963)
8	Weald clay	Brooker (1964)
9	Chicago clay	Brooker (1964)
10	Yarmouth clay	Bishop (1958)
11	Goose lake flour	Brooker (1964)
12	Kristiansand clay	Simons (1958)
13	Newfield clay	Namy (1970)
14	Oslo marine clay	Simons (1958)
15	Leighton Buzzard sand	Cole (1967)
16	Boston clay	Ladd (1965)
17	Sand	Simons (1958)
18	Belgian sand	Wade (1963)
19	Chew stoke clay	Bishop (1958)
20	Scott clay	Ladanyi et.al (1965)
21	Sand (loose)	Bishop (1958)
22	Sand (dense)	Bishop (1958)

FIG. 6·21 : VALUES OF K_o FOR A WIDE VARIETY OF SANDS AND NORMALLY CONSOLIDATED CLAYS [AFTER WROTH, 1972]

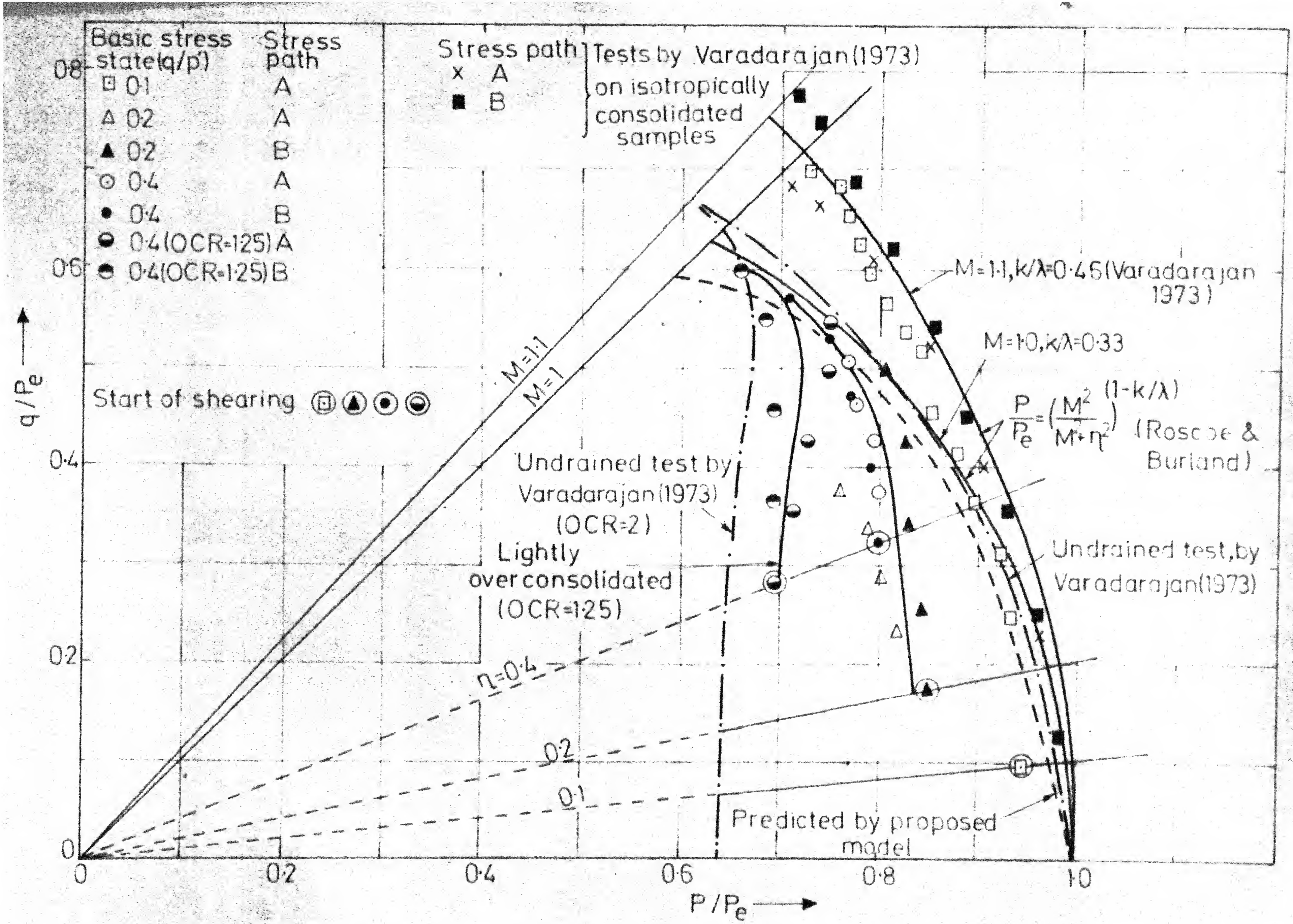


FIG.6·22 PREDICTED STATE BOUNDARY SURFACE FOR RANN OF KUTCH CLAY

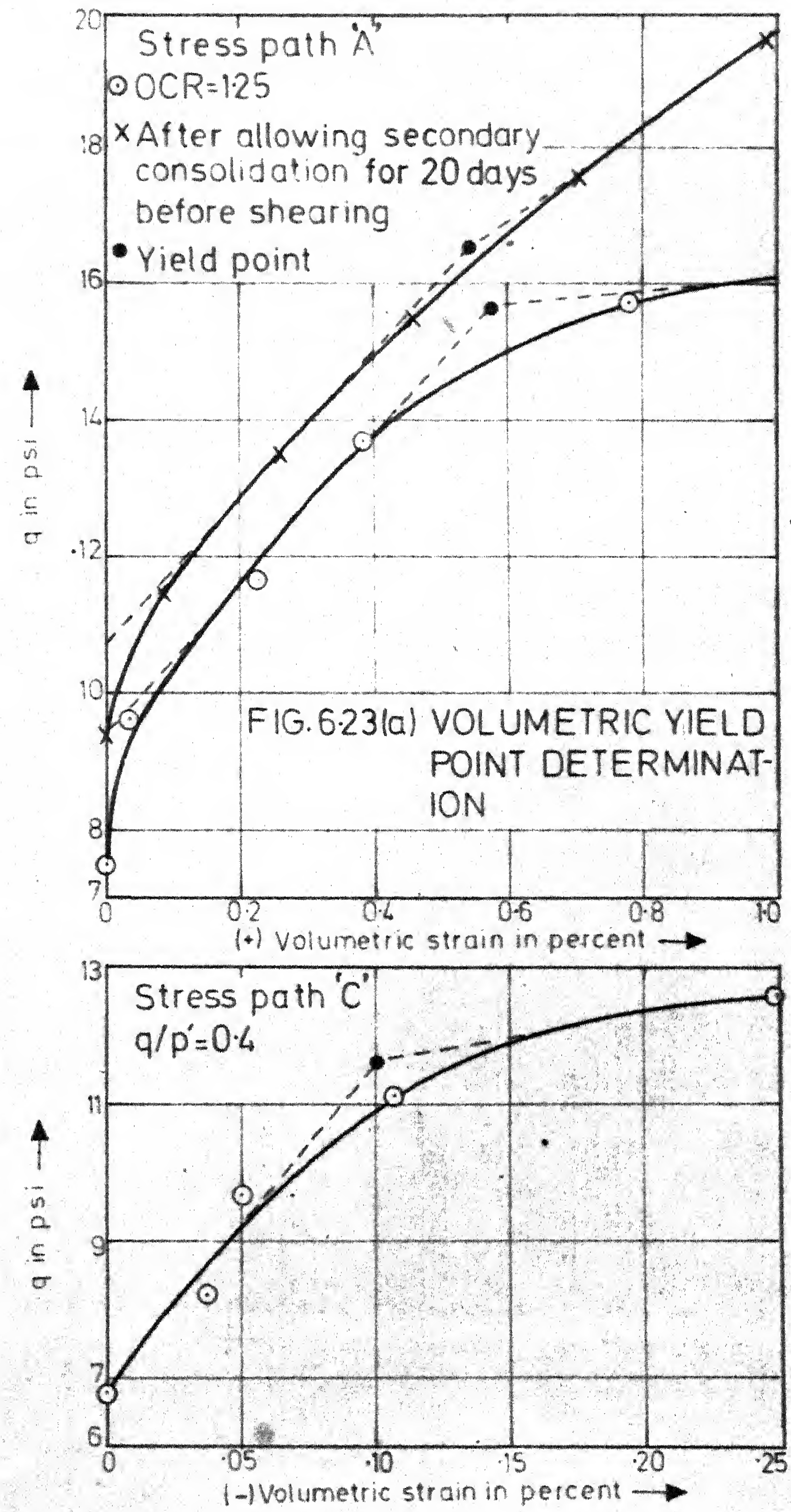


FIG. 6·23(a) VOLUMETRIC YIELD POINT DETERMINATION

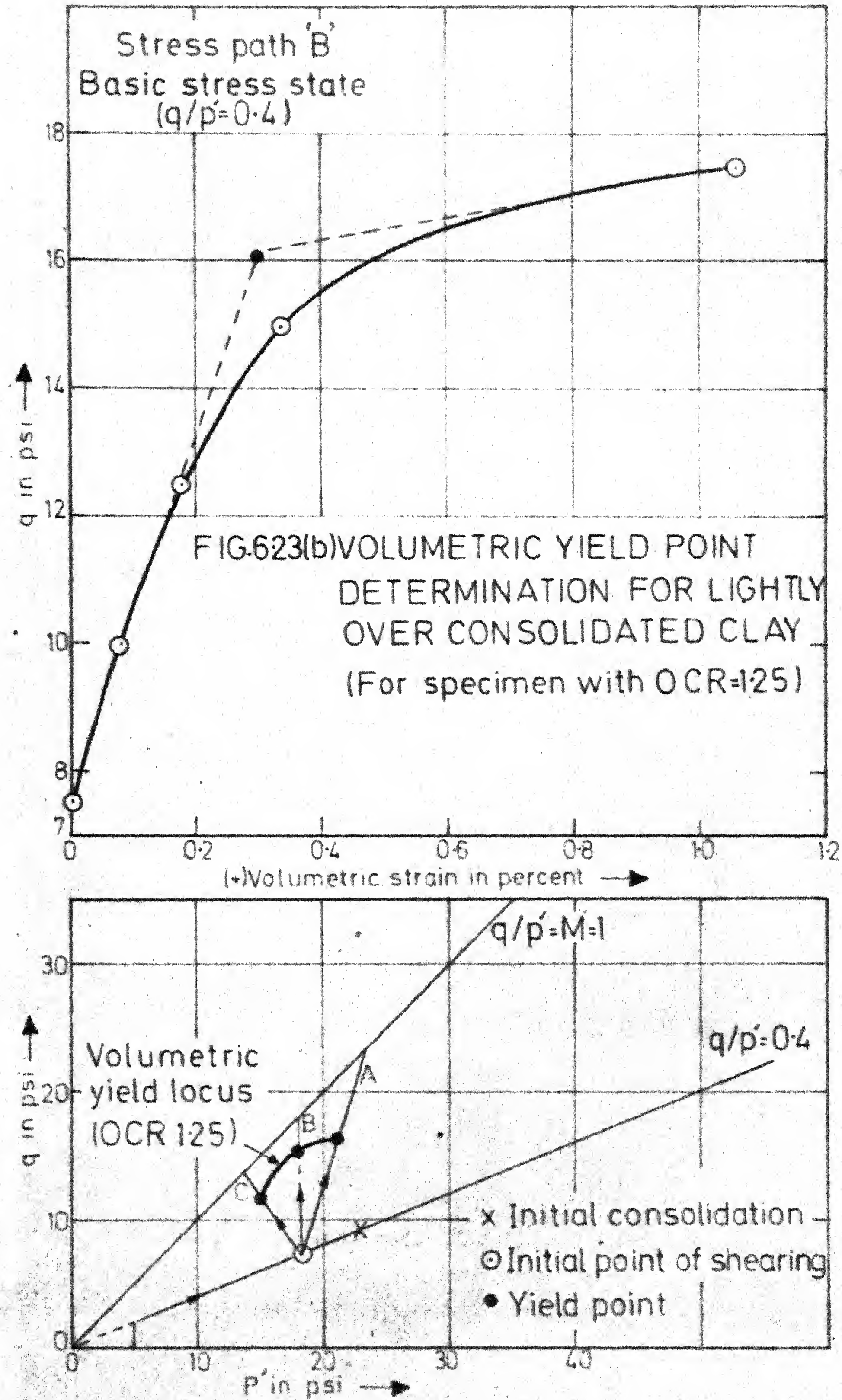


FIG.6.23(b) VOLUMETRIC YIELD POINT DETERMINATION FOR LIGHTLY OVER CONSOLIDATED CLAY

(For specimen with OCR=1·25)

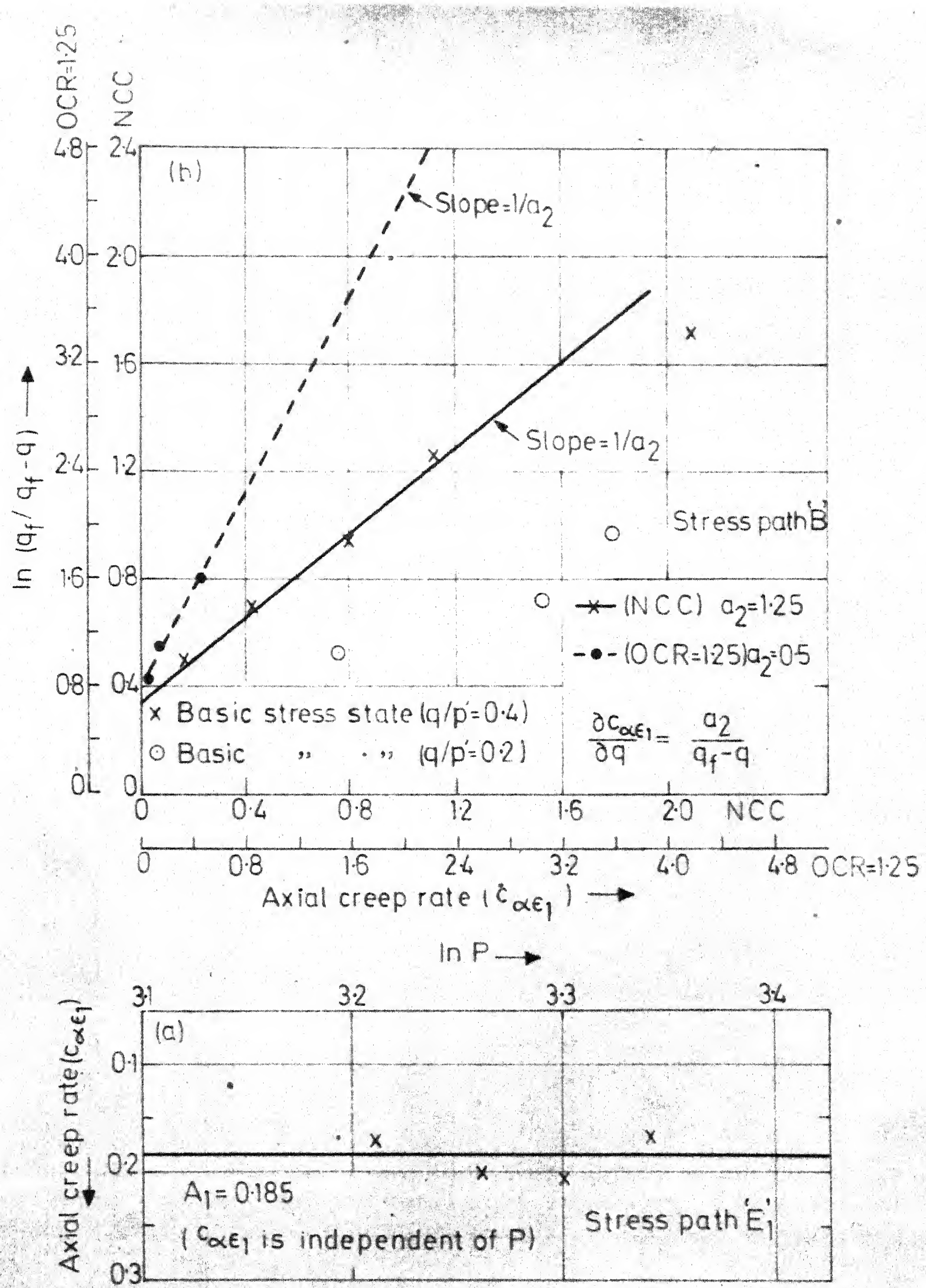
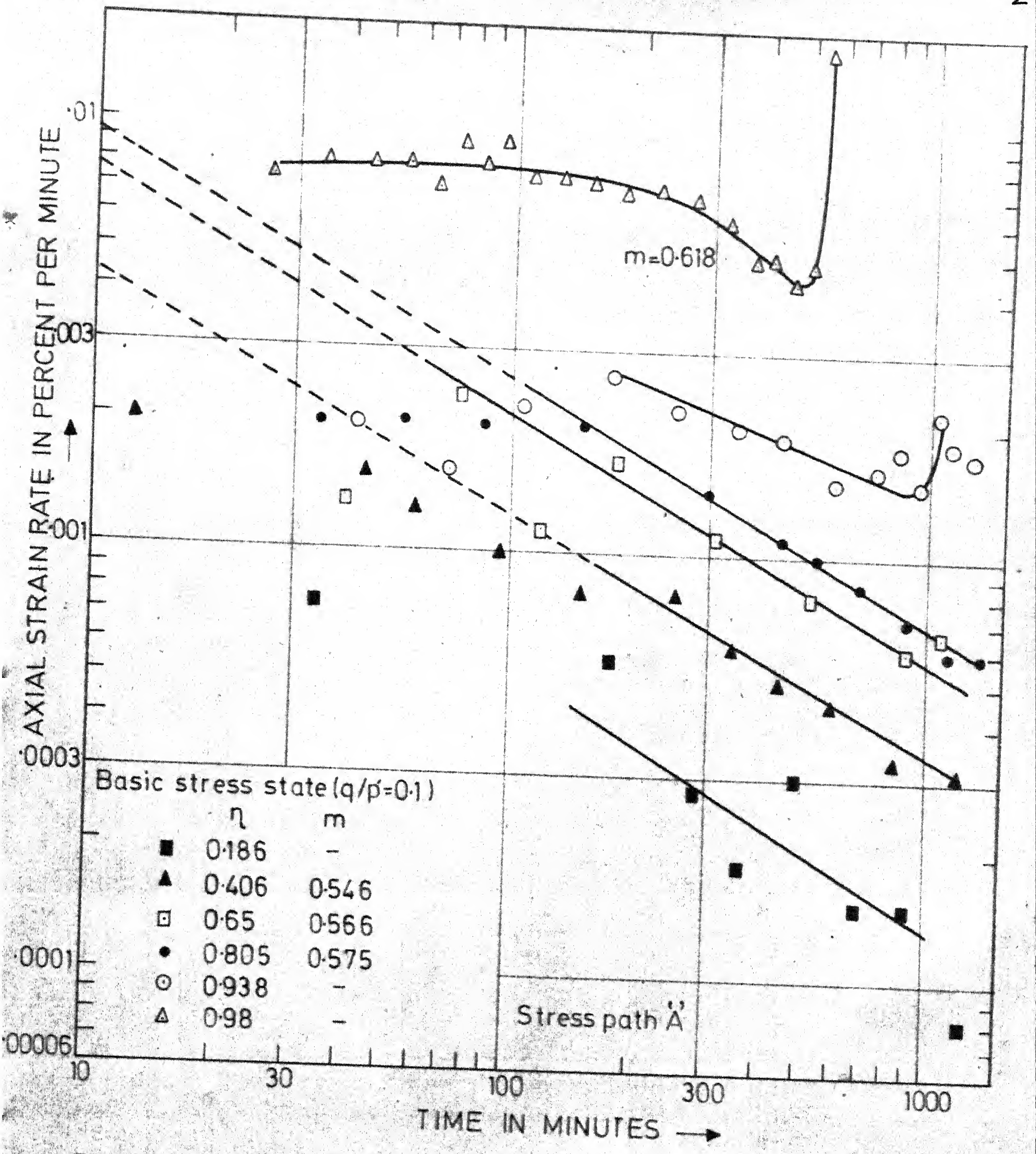
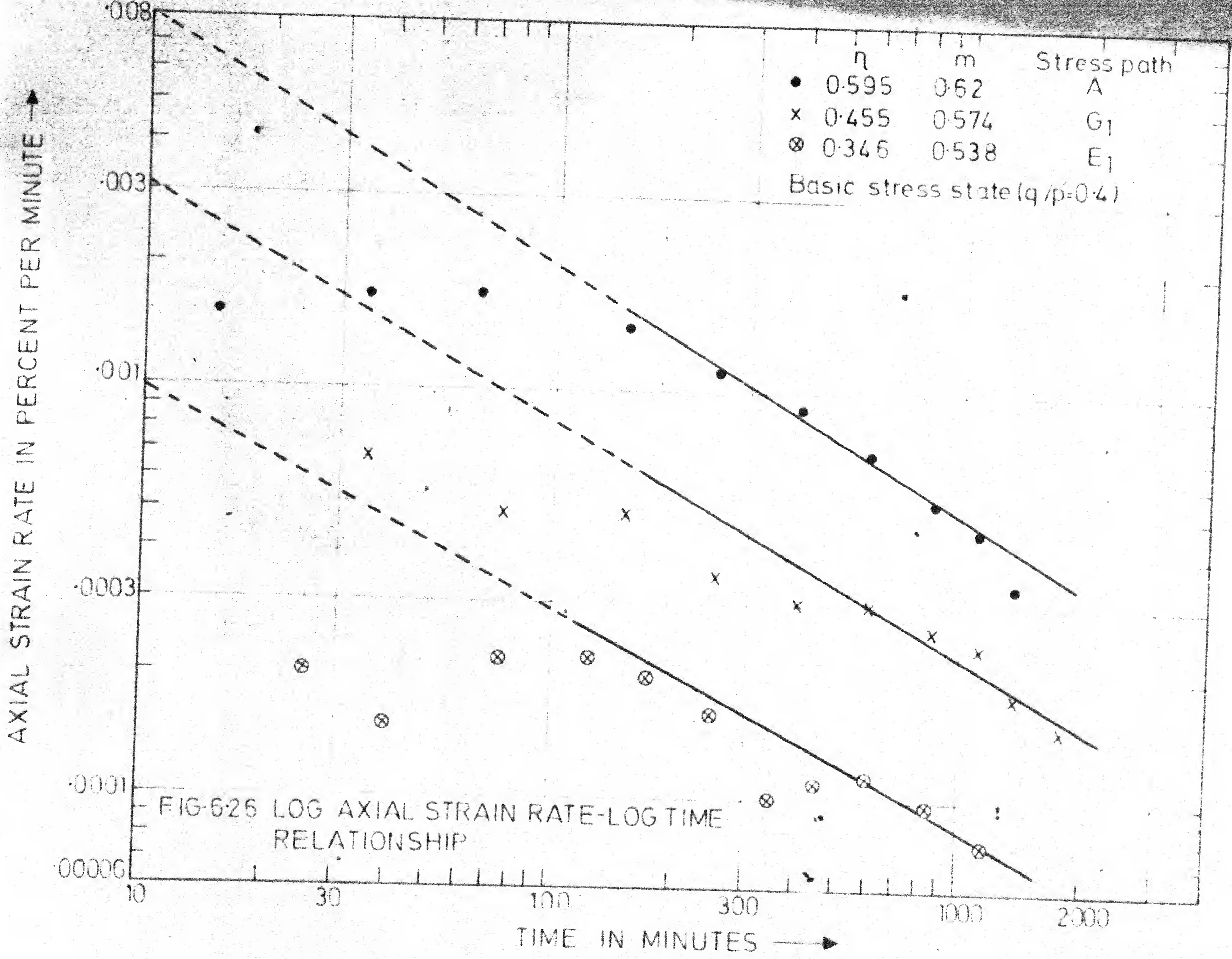


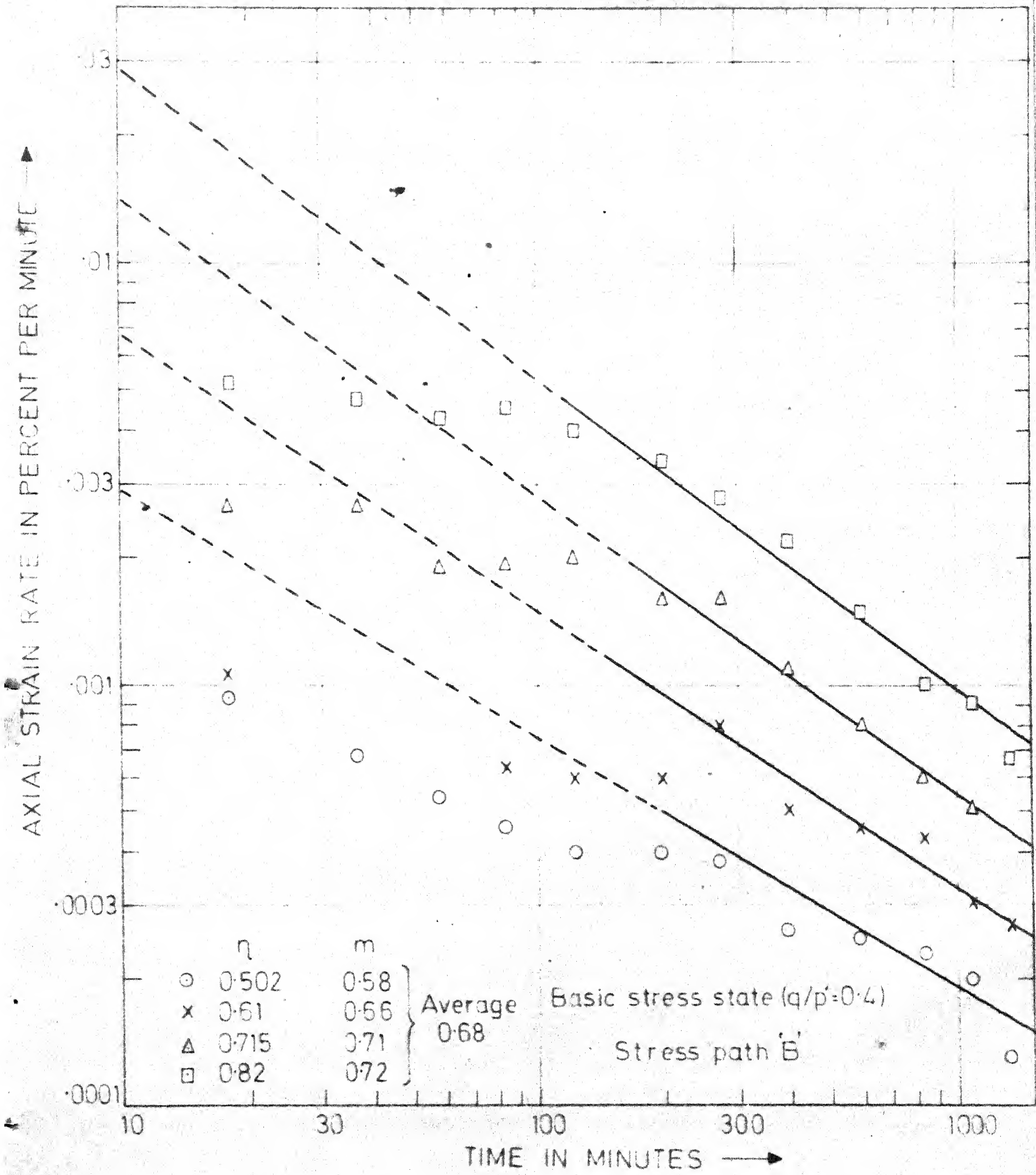
FIG.6·24 DETERMINATION OF PARAMETERS FOR 'B' AND 'E$_1$' STRESS PATHS FOR THE PROPOSED MODEL



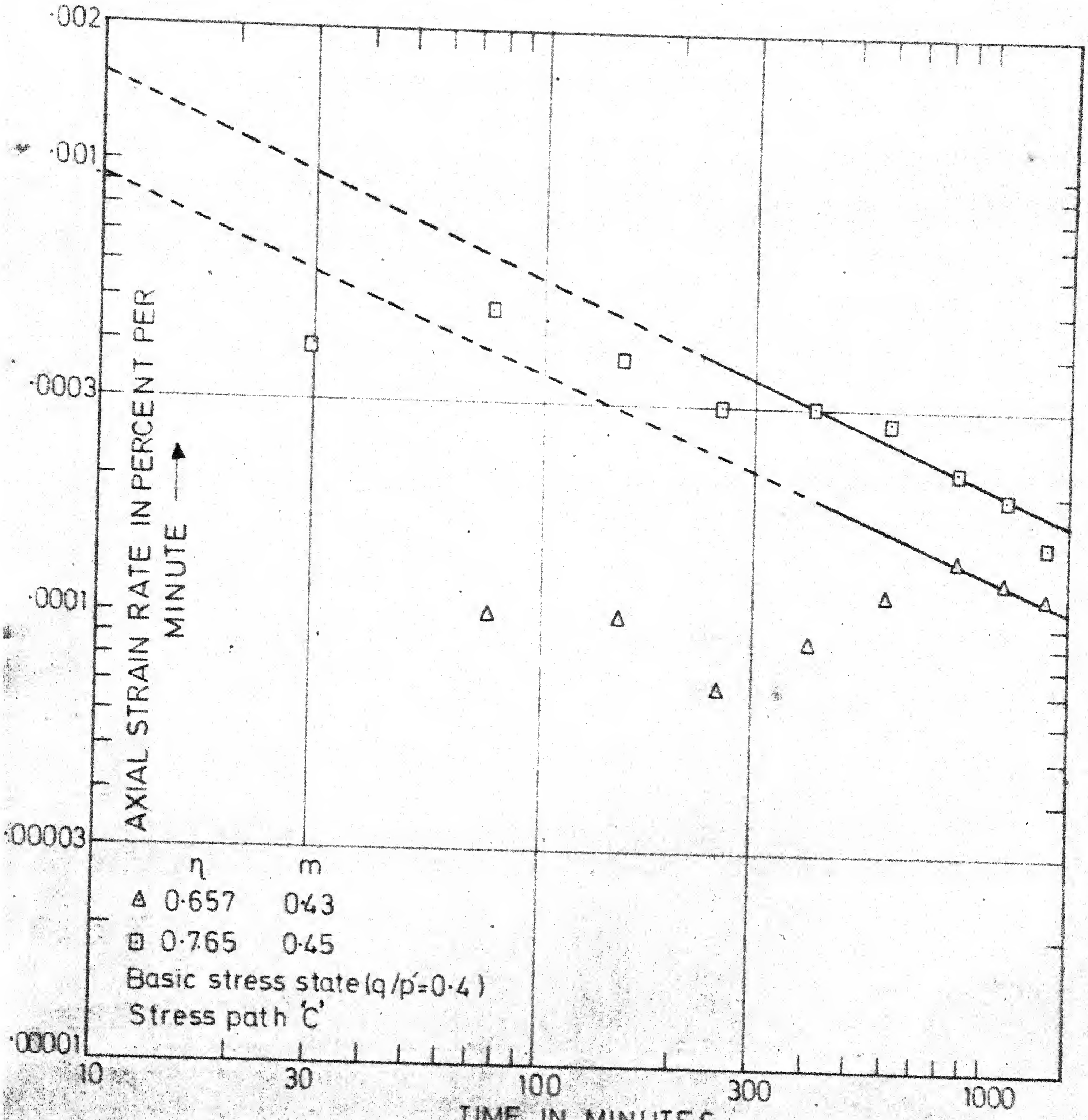
FIG·6·25 LOG AXIAL STRAIN RATE–LOG TIME RELATIONSHIP



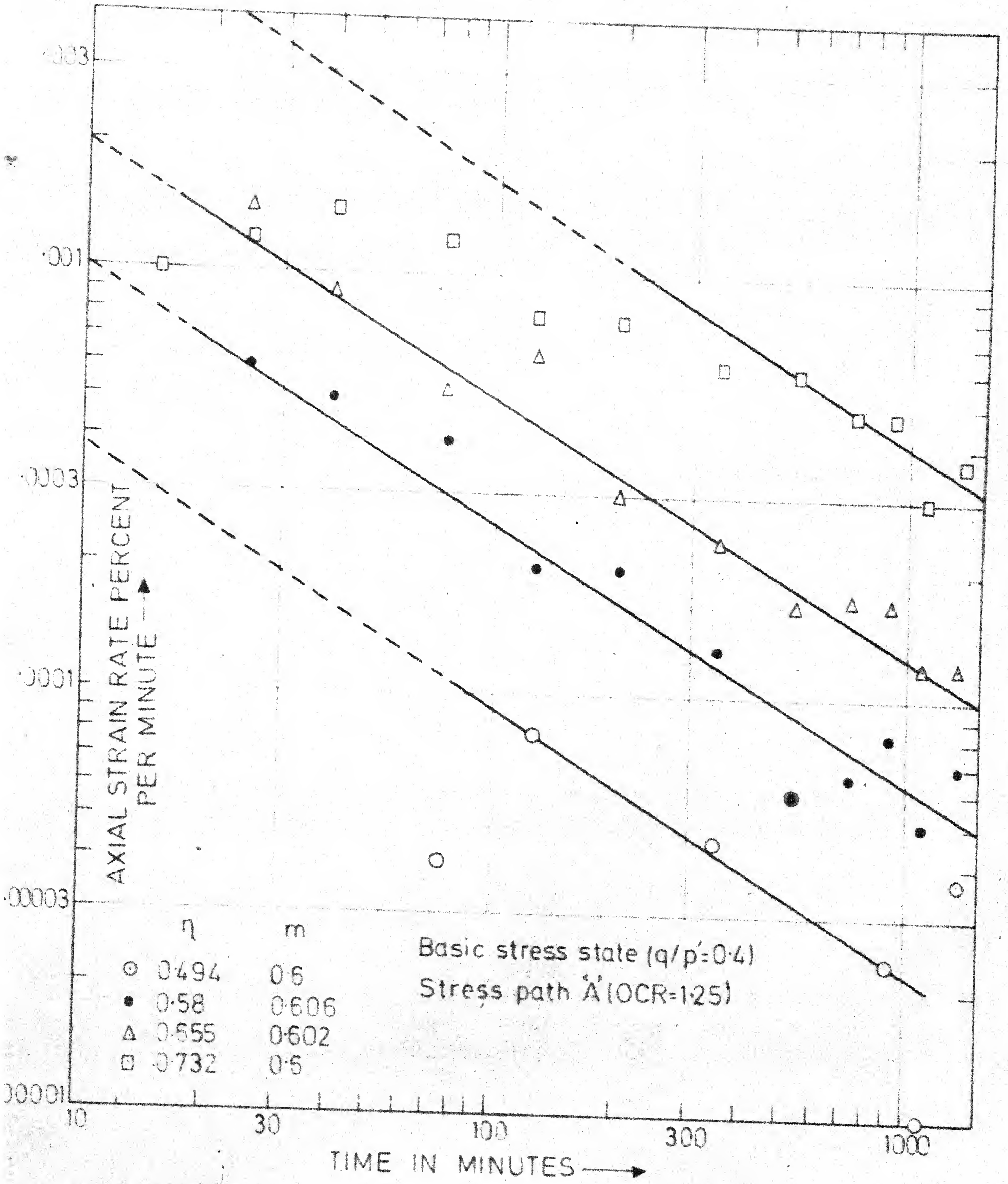
FIG·6·26 LOG AXIAL STRAIN RATE-LOG TIME RELATIONSHIP



FIG·6·27 LOG AXIAL STRAIN RATE - LOG TIME RELATIONSHIP



FIG·6·28 LOG AXIAL STRAIN RATE-LOG TIME RELATIONSHIP



FIG·6·29 LOG AXIAL STRAIN RATE - LOG TIME RELATIONSHIP

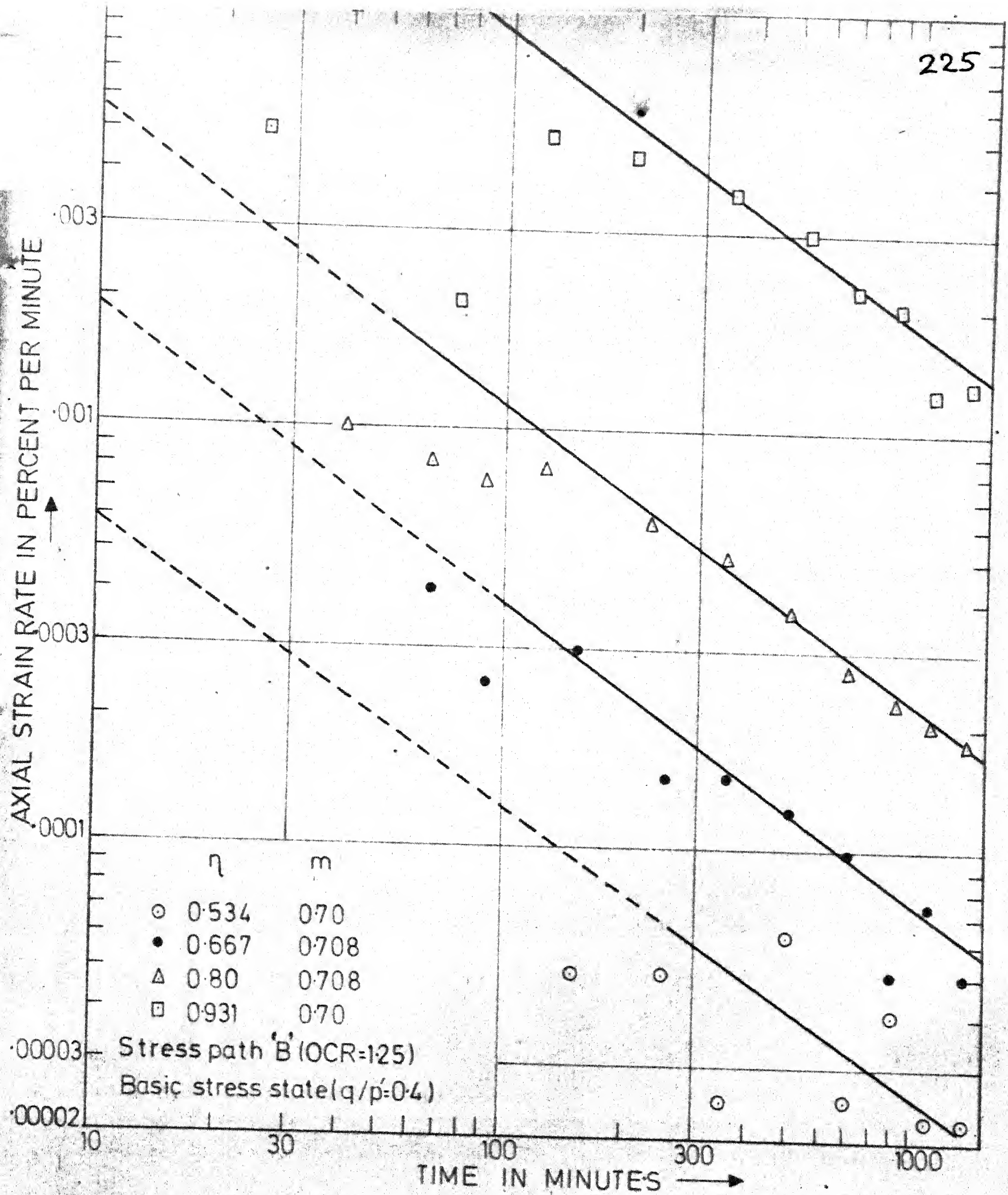


FIG.6·30 LOG AXIAL STRAIN RATE - LOG TIME RELATIONSHIP

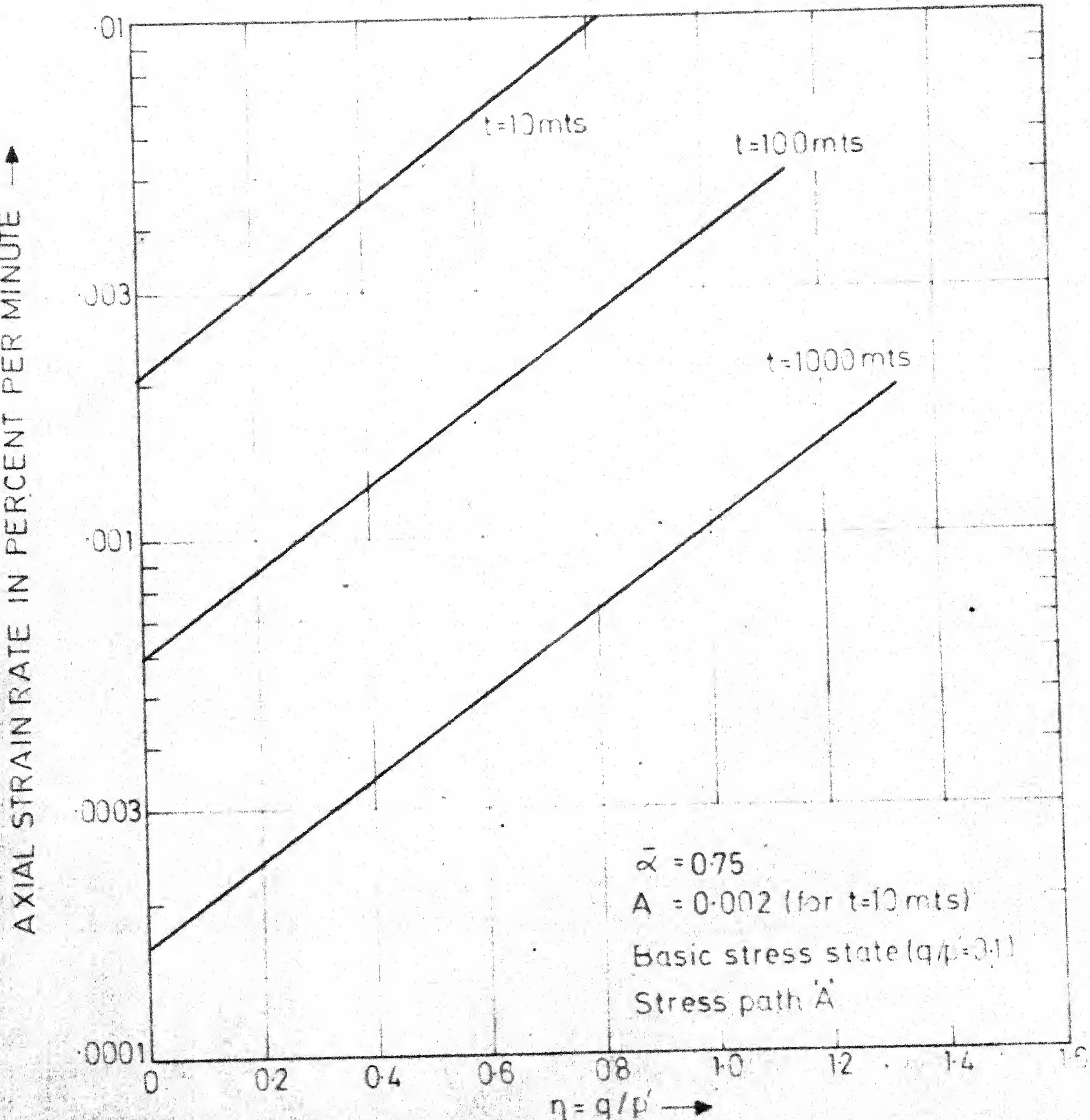
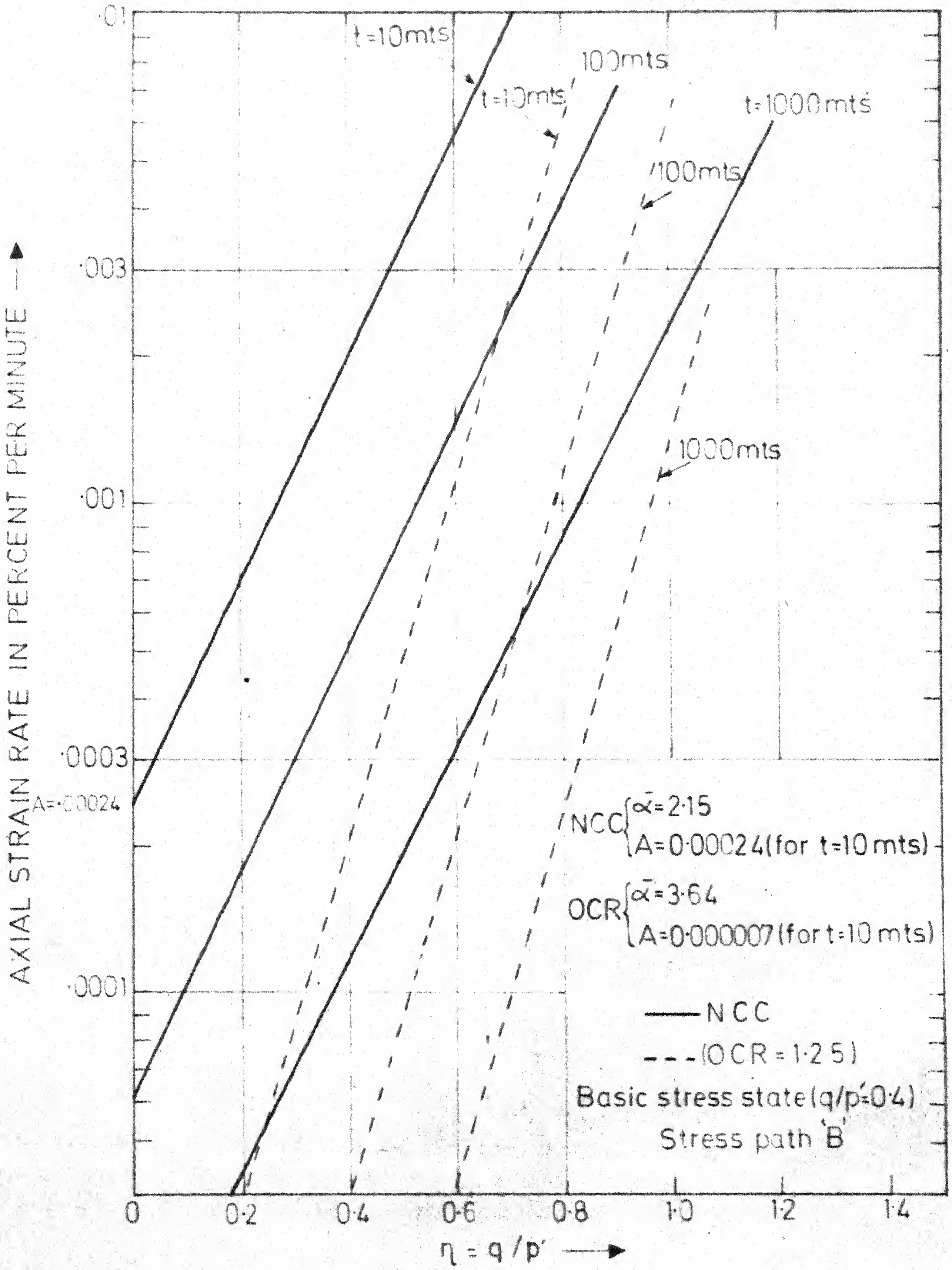


FIG. 6.31 LOG AXIAL STRAIN RATE - STRESS RATIO RELATIONSHIP



FIG·6·32 LOG AXIAL STRAIN RATE-STRESS RATIO RELATIONSHIP

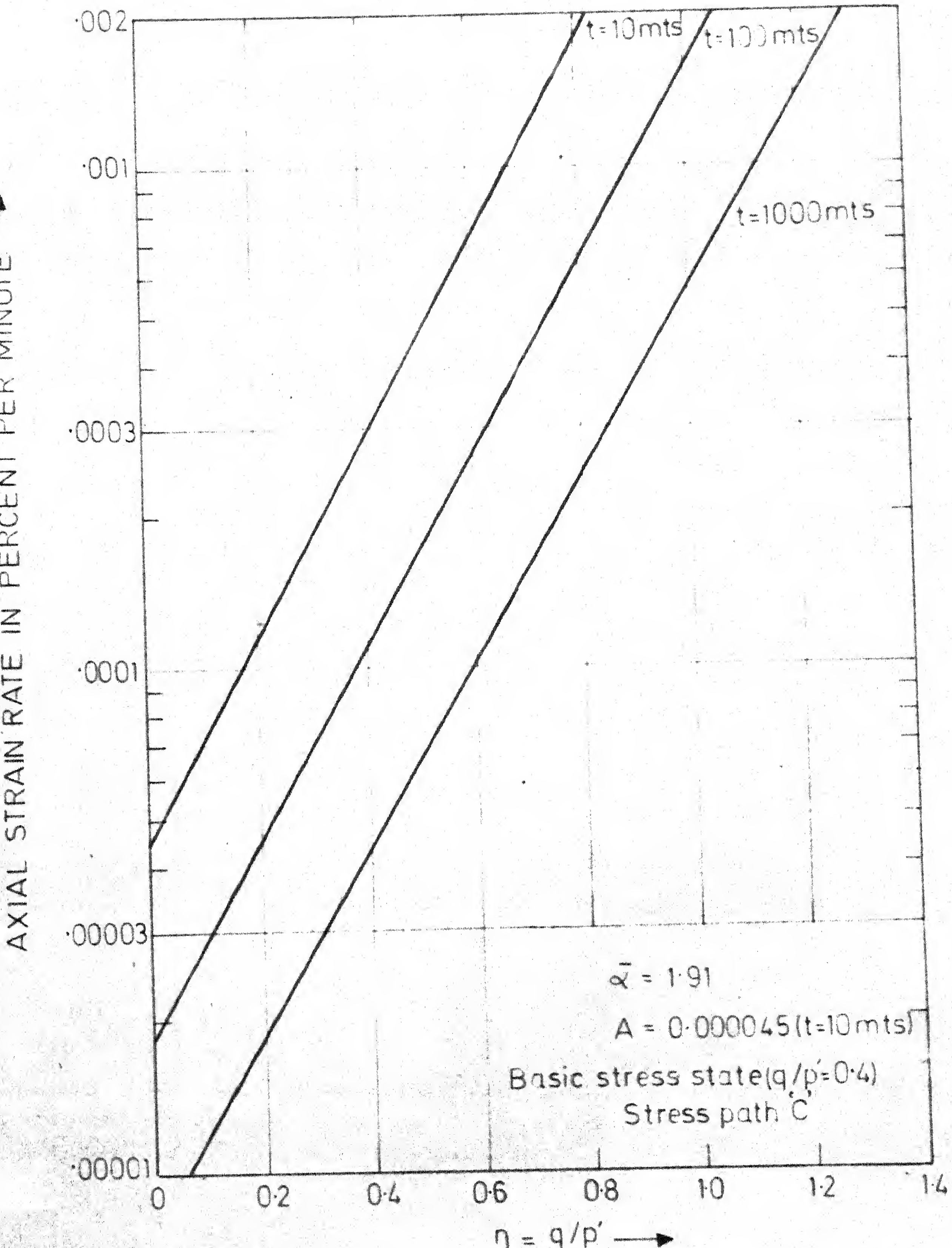
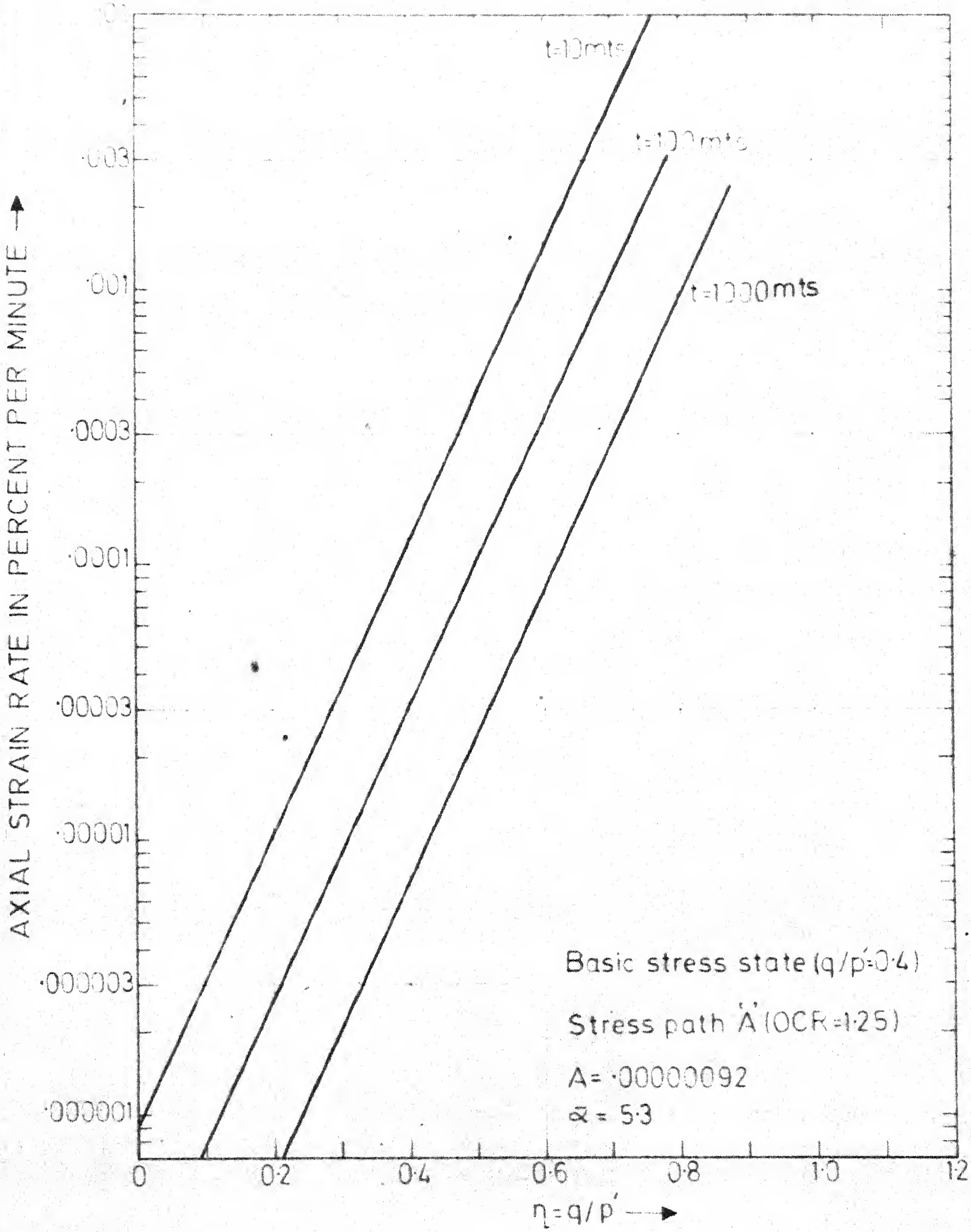


FIG. 6·33 LOG AXIAL STRAIN RATE-STRESS RATIO RELATIONSHIP



FIG·5·34 LOG AXIAL STRAIN RATE- STRESS RATIO RELATIONSHIP

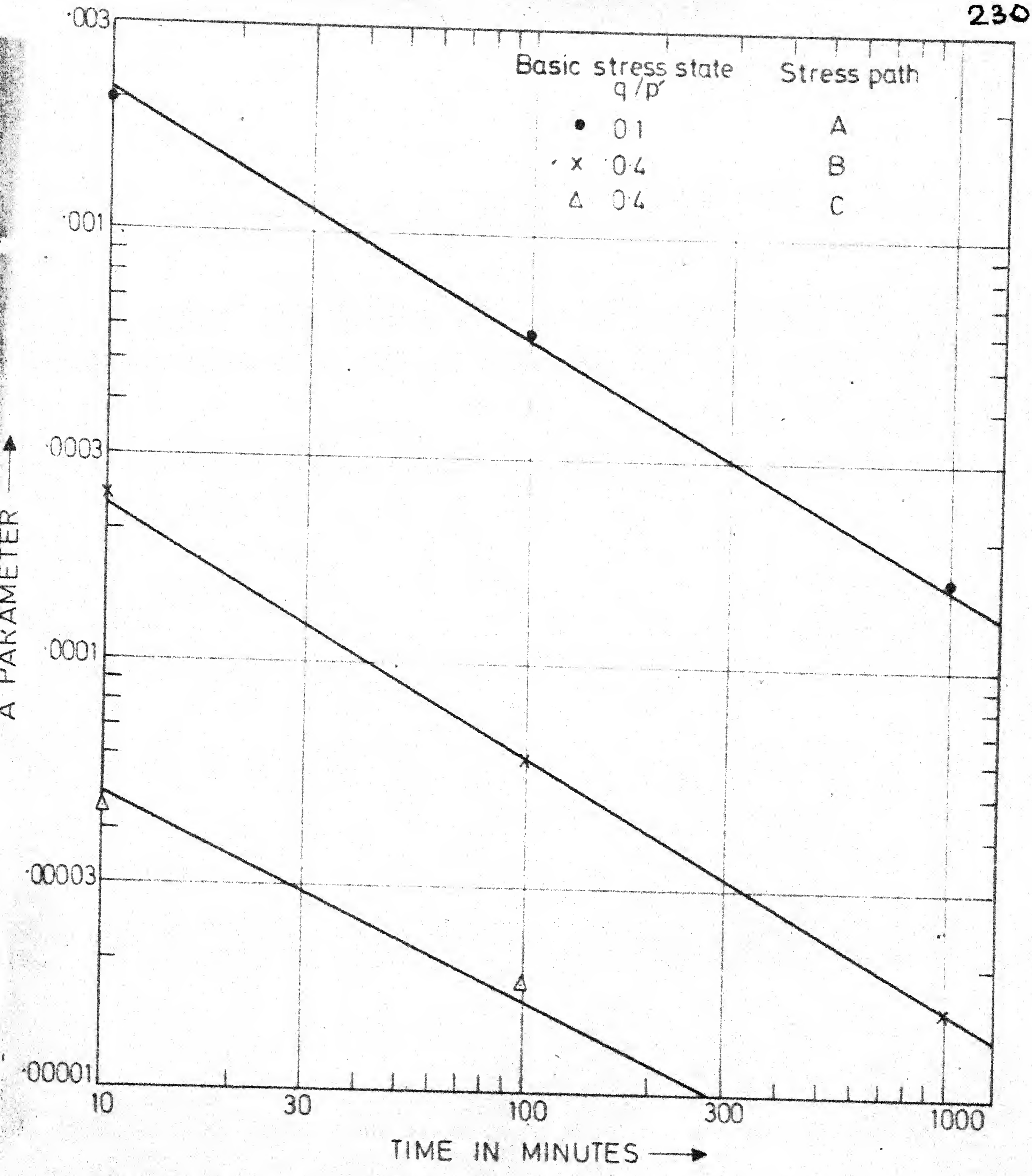


FIG. 6·35 VARIATION OF 'A' PARAMETER WITH TIME

Conventional drained tests on a sample, which had developed this critical pressure, showed a much stiffer stress-strain response. Tests on lightly overconsolidated samples appear to atleast qualitatively represent the insitu behaviour of the marine clay.

The unloading behaviour along various stress paths studied, showed that the recovery in volumetric and shear strains was very small. The magnitude of the probe has a substantial effect on the strain response.

Drained modulus is very much a function of the stress path. The effect on drained moduli of the previous stress history (isotropic/anisotropic consolidation; normally/lightly overconsolidated) is considerable and it is emphasized that the relevant consolidation stress history must be duplicated in the laboratory for any sensible results.

It is pointed out tnat the use of elastic theory in interpreting the results of some stress paths lead to an anamolous behaviour. This is due to the fact that tne theory of elasticity does not handle the stress path dependent, dilatant behaviour of the clay.

7.2 DRAINED CREEP BEHAVIOUR OF ANISOTROPICALLY CONSOLIDATED CLAY

Time readings for all the incremental creep tests along various stress paths were recorded for twenty four hours except for the K_0-consolidation test for which the readings were recorded for a

fortnight. Drained creep behaviour has been presented in the form of percent strain (volumetric, shear & axial) variation with log time (minutes) both for normally and lightly overconsolidated samples. Following conclusions were drawn.

The curves, after an initial portion upto a few hundred minutes resolve into linear strain-log time plots both for normally and lightly overconsolidated samples for various stress paths. Logarithmic creep rates (for A and B stress paths) observed for lightly overconsolidated clay are much smaller, compared to the corresponding values for the normally consolidated clay. The conventional drained test for a twenty day aged sample showed similar behaviour. Hence drained creep tests on lightly overconsolidated samples may atleast qualitatively represent the insitu creep rates.

The results clearly indicate that the drained logarithmic creep rates depend very much on the stress path. For the same stress ratio, the variation in creep rates for different stress paths is very significant.

Axial and Shear logarithmic creep rates (for A and B stress paths) show a linear increase with the stress ratio η upto an "yield value", beyond which the axial and shear creep rates become excessive leading to creep rupture. Test results of stress path E_1 (q-constant test) showed that the axial and shear creep rates

remain almost independent of η and confining pressure.

Volumetric creep rates are independent of η upto the "yield value", beyond which a decrease in creep rates occurs. Volumetric creep rates, with the limited data available in this study, suggest path dependence, although further detailed investigation is required to clarify this point.

7.3 PREDICTION OF THE OBSERVED STRESS-STRAIN-TIME BEHAVIOUR.

It is proposed that the stress-strain behaviour of a saturated clay along any stress path, may be predicted from the results of two basic tests (i) consolidation test from a k_o-line (q-constant) and (ii) pure shear test (P-constant) from a k_o-line. Evaluation of constants in the proposed model was done using results of E_1, B, E, B-unloading stress path tests. Predictions from this model have been compared with those from other models available in literature. It has been shown that for all loading stress paths, the predictions are in very good agreement with the experimental data. In case of unloading stress paths, the predictions are not as good and the discrepencies are likely to be due to the lack of appropriate and complete test data. The model is able to give atleast the right trend for these stress paths.

It has been suggested that the model can give reasonable predictions of k_o value and unloading behaviour during shear. The values

of σ_3'/σ_1' during k_o-unloading can also be estimated. The flow rule as obtained here is almost identical with the findings of Roscoe & Burland and Newland.

A model based on the results of P-constant and q-constant creep tests on k_o-consolidated samples, has been proposed to predict the creep rates over the whole range of η values upto failure for a variety of loading stress paths. A procedure has been outlined to evaluate the constants to be used in this model. It has been shown that $C_{\alpha v}$ is constant with η upto a "yield value", and thereafter it decreases to very small values at failure. A slight dependence of $C_{\alpha v}$ on stress path has been indicated. This at the present time (1975) clears the confusion regarding $C_{\alpha v}$ as pointed out in the next. This 'yield value' is corresponding to the value of η at which excessive axial creep rates develop. It has been shown that the decrease in $C_{\alpha v}$ begins at the same η value. Prediction of drained creep rates for the two loading stress paths is very good. The suggested semi-empirical approach is shown to be highly versatile and can be extended to the understanding of the stress-strain-time behaviour under more general stress conditions.

It has been suggested that the proposed model can be used to calculate settlement of footings on soft clays by adopting the Lambe's stress path approach. The same is applicable for drained creep rates also.

The Berkeley rate process model is shown to predict the steady state drained creep rates provided an average 'm' value and $\bar{\alpha}$ & A as functions of stress path and stress history are used in the relationship.

7.4 SUGGESTIONS FOR FURTHER RESEARCH

The following suggestions are made for further research:

In the study of stress-strain behaviour, more tests on lightly overconsolidated samples (for different OCR values) should be conducted along various stress paths in order to assess the insitu behaviour of soft clays.

Detailed laboratory tests to determine the variation of λ, K and M with stress ratio, pressure etc. should be conducted to assess the predictability of various Cambridge models.

Data from complete drained creep tests along various stress paths is needed to substantiate some of the observations made.

Complete stress controlled tests on k_o-consolidated samples should be conducted upto failure, as suggested, in order to evaluate the parameters needed for the proposed model.

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BIOGRAPHICAL SKETCH

The author was born in Agra (U.P.) on July 12, 1941. He did his undergraduate studies at the Institute of Technology, Banaras Hindu University, Varanasi and was awarded B.Sc. (Engg.) in Civil and Municipal Engineering in 1964. Since July 1964, he has been on the faculty of Civil Engineering, Institute of Technology, B.H.U., as Lecturer. He pursued his graduate studies at B.H.U. and was awarded M.Sc. (Engg.) in Civil Engineering in 1970. He was deputed to the Indian Institute of Technology, Kanpur for the doctoral programme under QIP in August, 1972.

The author is married to Meera and has a daughter, Monika, and a son, Sushant.

GRADE REPORT

DOCTOR OF PHILOSOPHY (Ph.D.)
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Year & Semester.	Course No.	Course Description	Units	Grade
1972-73 First	CE 637	Applications of Soil Mechanics	12	A
	CE 697	Special Topics in Soil Mechanics	12	S
	CE 699	Research	12	S
	M 605	Engg. Mathematics	12	A
1972-73 Second	CE 635	Soil Dynamics	12	A
	CE 641	Instrumentation Methods in Civil Engg. Practice	12	A
	CE 734	Shear Strength of Soil	12	A
	CE 699	Research	12	S
1972-73 Summer	CE 699	Research	24	S
1973-74 First	CE 699	Research	36	S
	CE 733	Analysis of Settlement of Soil	12	A
1973-74 Second	CE 699	Thesis		
1973-74 Summer	CE 699	Thesis	24	S
1974-75 First	CE 699	Thesis	48	S
1974-75 Second	CE 699	Thesis	48	S
1974-75 Summer	CE 699	Thesis	24	S

C.P.I. = 10.00